2.2 SITE INVESTIGATION

2.2.1 The Soil Investigation

2.2.1.1 Origin of Soil (Weathering, Erosion and Deposition)

Soil is formed by weathering of rocks. Weathering is a destructive process by which the rock surface exposed to the atmosphere is broken down into smaller particles, the size varying from a large size boulder to a small particle of clay. A rock may weather either physically or chemically.

Physical Weathering

In the physical process, a rock may weather by the action of wind, water or temperature variation and disintegrate by alternate freezing and thawing in the cracks in the rock. This action results in soil particles to retain the same composition as that of the parent rock. The particles formed by such action are approximately angular, sub-angular, sub-rounded, rounded or well rounded.

Chemical Weathering

In chemical weathering, the structural arrangement of the minerals in the parent rock is changed partially or completely. Water, oxygen and carbon dioxide are the predominating agents and they play an important role in the process. Chemical weathering results in the formation of a group of particles of colloidal size known as clay minerals. Most of the clay minerals resemble the shape of a plate having a high surface area to mass ratio with the result that surface forces influence their properties significantly. The various processes by which chemical weathering takes place are oxidation, hydration, hydrolysis and leaching.

2.2.1.2 Types of Soil

Soil in group exhibits similar behavior in a given engineering situation. A classification system also provides a common language to engineers and builders for the exchange of information and experience regarding soils. Soils are classified differently in different disciplines.

An engineering soil is an un-cemented or very weakly cemented accumulation of particles, the sizes of which range from a few micron (colloidal) to a few meters (cobbles and boulders).

Soils are given different names depending on the medium of transportation and deposition. The most important groups of soils are described below:

Residual Soil : The soil formed by the weathering of the underlying rock is called Residual soil or also called eluvial soil. The weathered product is deposited at its place of origin, i.e., not transported by any transporting agent. Such soils are formed on weak and low gradient slopes into which rain water penetrates deeply leading to weathering of the rock or material. According to its nature and the climatic conditions, such types of soils are more or less thick but may reach a major thickness in certain conditions. If the thickness of the residual soil is thin, the site must be considered as a rocky site. The residual soil is generally homogeneous in nature. The original rock fabrics may still be intact. The original rock layers may be present. Mostly near the surface, these soils contain small rock debris. The grain size of this debris frequently increases with depth below the surface. As the nature and composition of the soil matrix depends on the nature of the parent rock, physical parameters such as cohesion, permeability, frictional angle and unit weight may vary greatly from one to another.

Colluvial Soil : Soil deposited after some movement or transportation from its place of origin is called colluvial or allogenic soil. Such type of soil is deposited after some movement or transportation from its place of origin. The transporting agent is mostly gravity. The soils are deposited after landslides, rockslides, falls, debris slides, and mudslides. Angular stones and blocks in a matrix of clay, silt, sand or gravel characterize such soils. The colluvial soil is generally thicker at the top of the slope. These types of soils are often unstable and should be considered with caution.

Physical parameters such as cohesion and frictional angle are highly dependent on the rock from which they originate and on the type of movement that has occurred.

Alluvial soil: Soil transported by water over considerable distances and then deposited is alluvial soil. The particles, which were angular at their various sources, are actively smoothened as a result of friction during transportation. Layers of course material deposition are the result of strong water current and heavy erosion, whereas fine-grained layers depositions are the result of quieter periods of water current. These alternate periods of high and low water current of the flowing water produces an alternation of coarse and fine weight of a given terrace which consequently varies from one layer to another. Layers of pebbles and boulders in a matrix of course sand are very pervious, non-cohesive and have a high angle of internal friction. Layers of fine materials such as sand, silt and a mixture of clay and silt are semi-peeious to impervious, highly cohesive and have a rather low angle of internal friction. The alluvial soil can further be classified as primary, secondary and tertiary depending upon the location as first, second and last stages of deposition respectively.

Morainic soil: Morainic soil is soil deposited at a place by glacial action. These soils are confined to relatively high altitudes in the Himalaya, though they may be present as far down as 2000 m and lower elevations. Morainic soils consist of gravel and pebbles within a clayey silt or sandy matrix. Transportation by glacier therefore gives particular shapes and marks to the particles. Smoothening is however not as complete as in the case of alluvial particles. Glacial deposit particles are sub-rounded or sub-angular. Often huge boulders (erratic boulders) appear within or on the surface of the morainic soils. These soils sometimes reach great thickness. They generally have poor grading and horizontal continuity, except when they have been reworked by stream action during the glacial period. As the morainic soils on the slopes are rather unstable, they should be surveyed and tested carefully.

Eoline: If the transporting agent is wind, the soil is called eoline. Eoline soil mostly consists of sand and silt sizes and are most common in deserts, sea beaches, etc.

The grading and roundness of the various soil types are shown in Table No.2.1.

Table No. 2.1: Grading and Grain Roundness of Geological Soils

Geologic Soil	Well Graded	Medium Graded	Poorly Graded	Grain Roundness
Alluvial terraces	Frequent	Rather rare	Rare	Well-rounded to rounded
Alluvial fans	Frequent	Rather frequent	Rare	Well-rounded to rounded
Debris flow layers in alluvium	Rather rare	Frequent	Frequent	Rounded to sub-rounded
Glacio-fluviatile	Rather frequent	Frequent	Rather rare	Rounded to sub-rounded
Glacial deposits	Rare	Rather frequent	Frequent	Sub-rounded to sub-angular
Colluvium (slope debris)	Extremely rare	Rare	Very frequent	Angular
Residual soils	Rare	Rather rare	Frequent	Sub-angular to angular

The engineering behavior of soils having different origins depends on their permeability, grading, anisotropy, and their capability to resist stress imposed by applied loads.

Permeability

A material which contains continuous voids is said to be permeable. Practically all materials such as granite, soils, neat cement, etc. are permeable to some extent, and allow water to flow through their interconnected voids. This property of the material is known as permeability. Coarse-grained soils are generally well drained if the soil does not contain impervious layers such as silt, fine sand or clay. The drainage capacity of poorly graded soils like colluvium can be high due to the presence of a large volume of voids and these soils are known to have high permeability.

However, weathering may plug the voids if the debris is from a rock that produces clay and fine material through alteration. In the course of flowing through such voids, water can exert tremendous pressure, which is often known as "Seepage Pressure". The seepage pressure has a decisive effect on the cost and difficulty of many construction operations. The constructions include excavation on the open cuts in water bearing sands, or on the rate at which soft clay consolidates on the influence of the weight of a superimposed fill, or on the safety of hydraulic structures. Thus it is very important to have a clear knowledge of the permeability characteristics of soil in order to be able to successfully solve problems related to disaster management.

Grading

In general, well-graded soil settles less as compared to poorly graded soils. This is due to the fact that in well-graded soil, finer soil particles fill the voids between coarser particles. Thus there are less voids, and consequently such soils have better resistance to load than poorly graded soils.

Anisotropy

Anisotropy is another factor that favors resistance of soil to load stresses. Anisotropy can be vertical or/and horizontal. Anisotropy is vertical when there is a major discontinuity such as a thick layer of fine material like fine sand, silt or clay interstratified into coarser material. With alluvium and fluvio-glacial deposits, there is vertical anisotropy. In a few instances, vertical anisotropy is associated with glacial deposits. The probability of this is less in the case of colluvium and residual soils. The anisotropy is horizontal when there is a change in the lateral extent of a deposit. This is frequently the case with colluvium, glacial and fluvio-glacial deposits, but happens less frequently in the case of alluvium and residual soils.

Resistance to withstand loads

Soils display characteristics of their parent rocks from which they are derived. For example, alluvial deposits, provided they are isotropic and exist in thick layers having constant granulometric properties, possess a high modulus of elasticity. On the other hand, clay soils such as residual soils, the mother rock of which is clayey rock, are plastic and have low resistance to load stresses. The plasticity characteristics of these geologic soils are more or less proportional to their clay content. This means that clayey soils are prone to settlement. The clay content is therefore a good indicator to determine whether geologic soils are resistant to applied loads or not. It can be said that alluvial soils containing a high percentage of boulders, cobbles, gravel and sand will resist better load stresses than geologic soils containing a high percentage of silt and clay. Table No. 2.2 given below lists the indications of different geologic soils in terms of anisotropy, grading, permeability, elasticity and resistance to sustain load stresses. A plus sign indicates positive influence and a minus sign indicates negative influence.

Cohesion

Cohesion is the property of soils by virtue of which one soil particle attracts an other soil particle in its immediate vicinity. This arises due to the presence of deficit of charges on the surface of fine to very fine-grained soils. This force is absent in sand because this type of soil does not have charge deficiency on the surface. However, by prolonged exposure to water and other weathering agencies granular particles may change into fine-grained soils and thus exhibit cohesive properties.

Table No. 2.3 given below provides a picture of the relationship between geologic soils and their cohesive characteristics. The table also shows how far the weathering process may transform geologic soils from non-cohesive to cohesive.

Table No. 2.2: Resistance to Load of Geologic Soils

Geologic Soil	Vertical anisotropy	Lateral anisotropy	Grading	Permeability	Nature of Material	Resistance to load in general
Alluvial terraces	May be important (-)	Weak (+)	Good (+)	High (-)	Elastic (+)	Good to excellent (if vertical anisotropy is weak)
Alluvial fans	Rather important (-)	Weak (+)	Moderate to good (~+)	High (-)	Elastic (+)	Good to excellent (if vertical anisotropy is weak)
Debris flow layers in river terrace	Weak (+)	Weak (+)	Poor (-)	High (-)	Elastic (+)	Moderate (due to poor grading)
Glacio-fluviatile	Rather important (-)	Rather high (-)	Moderate to good (~+)	Moderate to high (~+)	± Elastic (~+)	Moderate (due to frequently high anisotropy)
Glacial deposits	Weak (+)	Moderate (+)	Poor to fair (-) (+)	Low to moderate (+)	±Elastic (~+)	Moderate (poor grading) to good
Reworked glacial mat. (creeping)	Weak (+)	Moderate (-) (+)	Poor to fair (-) (+)	Moderate to high (+)	Low elasticity, unstable (-)(+)	Low due to instability of the material
Colluvium (slope debris)	Rather weak (~+)	Rather weak (~+)	Poor (-)	High (low when clayey through weathering) (-) (+)	Moderately elastic (low elasticity when clayey through weathering) (-) (+)	Moderate (poor grading)
Residual soils	Weak (+)	Weak (+)	Poor (-)	Poor to high depending on mother rock (-) (+)	Plastic to moderately elastic (-)	Rather weak

+ Positive Influence **-** Negative Influence

2.2.1.3 Soil Classification in Civil Engineering

The principle terms used by civil engineers to designate soils are gravel, sand, slit and clay. Particle sizes in soils can vary from 100 mm to less than 001 mm. Depending on the particle size, soils in civil engineering are designated as boulder, cobble, gravel, sand, silt or clay.

In civil engineering, soils are put into groups in which all the soils in a particular group have similar characteristics. The soils in each group are denoted with a group symbol. In addition to the group symbol, a general description should include the color of the soil and the details of the predominant particle shape. The soil in-situ may be homogeneous or stratified. Sands and gravel may be described in terms of their slate of compactness. Clays may be described in terms of their consistency. A general rule of thumb to describe soil consistency in the field is presented in Table No. 2.4 given below.

Table No. 2.3: Cohesion of the Main Geologic Soils

Geologic soil	Cohesion		Change from non-cohesive to cohesive due to weathering
Alluvium	Non-cohesive		None or very rare
Colluvium	Non-cohesive	Might be cohesive when weathered	May occur when debris is from clay rocks like clay-stones, marls, slates, phyllites or schists
Glacial deposits	Non-cohesive in general	Some cohesion in specific but rare types	None or very rare
Glacio-fluviatile	Non-cohesive in general		None or very rare
Residual soils	May Non-cohesive	May Cohesive	Cohesive residual soils have a clay rock as the mother rock or a rock bearing minerals that may degrade into clay, for example, granite and gneiss

It is also important to describe the arrangement of minor geological details present in the soil as a whole as these details can influence the engineering behavior of the soil to a considerable extent. Features such as layering, fissuring and inclusions should be described. Examples of soil descriptions are given below.

a. Coarse-Grained Soils

Coarse-grained soil is yellowish grey, medium, homogeneous, poorly graded clean sand consisting of sub-angular particles.

Table No. 2.4: Consistency from Field Inspection

Consistency	Field indications
Very stiff	Brittle or very hard.
Stiff	Cannot be molded with the fingers.
Firm	Can be molded with the fingers by strong pressure.
Soft	Easily molded with the fingers.
Very soft	Exudes from between the fingers when squeezed in the fist.

b. Fine-Grained Soils

Light grey, firm, silty clay of low plasticity with small fissures and silt inclusions. The soil classification systems that are in general use in civil engineering are:

1) Grain size classification.
2) Unified soil classification.

c. Grain Size Classification

This classification is based on grain size only. The size ranges for these types of soils are shown in Table No.2.5 below.

Table No. 2.5: Grain Size Classification

S.N.	Soil Type	Range of Sizes in mm	
		Maximum	Minimum
1	Coarse Sand	4.75	0.60
2	Medium Sand	0.60	0.20
3	Fine Sand	0.20	0.075
4	Coarse Silt	0.075	0.02
5	Medium Silt	0.02	0.006
6	Fine Silt	0.006	0.002
7	Clay	Less than 0.002	

In Table No. 2.5 above, the terms sand, clay, etc. are used only to describe the sizes of the particles between specified limits. The same terms are also used to describe particular types of soil. In a given soil, if the proportion of sand-size particles is less than that of clay-sized particles, the soil may be described as sandy clay. All clay-sized particles are not necessarily clay mineral particles, i.e., the finest rock flour particle may be of clay size.

The behavior of gravel and sand sized particles is entirely different from that of clay and silt particles. Hence, in civil engineering, in broad terms gravel and sand are referred to as Coarse-Grained or Non-cohesive soils, and silts and clays are referred to as Fine-Grained or Cohesive soils. The size which demarcates the boundary between coarse-grained and fine-grained soils is 0.075 mm.

d. Unified Soil Classification System

The unified soil classification system was first developed by Cass Grande and adopted in 1942 by the Corps of Engineers of the United States of America. This system of classification is the most popular one and is universally accepted for classification of soils for general engineering purposes.

The system uses both particle size and plasticity characteristics of soil. Furthermore, the system includes field identification of soils. In this classification system, the group symbol consists of primary and secondary descriptive letters. These letters and their meanings are given below:

Primary letter

G : gravel
S : sand
M : silt
C : clay
O : organic
P_t : peat

Secondary letter

W : well-graded
P : poorly-graded
M : non-plastic fines
C : plastic fines
L : low plasticity
H : high plasticity

The essential features of the unified soil classification system are shown in Table No.2.6. Normally, combinations of two symbols are used to classify a soil, for instance, GP, SW, CL, etc. They can also be combined as double symbols to classify mixed soils, such as GC-CL.

<u>Coarse-Grained Soils</u>

If more than 50% of the soil is of sand size, it is designated as coarse-grained soil. There are eight groups of coarse-grained soils (Table No. 2.6). Coarse-grained soils are sub-divided into gravel and sand. The soil is designated as gravel if more than 50% of the coarser material is of gravel size, i.e., it is retained on a 4.75 mm sieve. On the other hand, the soil is classed as sand if more than 50% of the coarser material is of sand size, i.e., it passes through a 4.75 mm sieve. If the coarse grained soil contains less than 5% fines and is well-graded, it is given the symbol GW and SW, and if the soils are poorly graded, the symbols GP and SP are given. On the other hand, if it contains more than 12% fines, it is designated as GM, GC, SM or SC are per the criteria given in Table No. 2.6. If the percentage of fines is between 5 –12%, dual symbols such as GW - GM or SP - SM are given.

<u>Fine-Grained Soils</u>

If more than 50% of the soil is finer than 0.074 mm size, the soil is designated as fine-grained soil. Fine-grained soils are also divided into two groups.

- Soils having a liquid limit of less than 50 (regarded as low compressibility or plasticity) which are given the symbol ML, CL or OL.
- Soils having a liquid limit of more than 50 (regarded as high compressibility or plasticity) which are given the symbol MH, CH or OH.

Independent of plasticity and liquid limit, mixed soils can also be combined as GC-CL.

<u>Organic Soils</u>

If the soil contains truly divided organic matter and is of fibrous nature, the soil is organic and is designated as peat (P_t).

Plasticity:

H	high plasticity	L.L.	>	50%
I	intermediate plasticity	L.L.	>	35-50%
l	low plasticity	L.L.	<	35%

Table No. 2.6: Unified Soil Classification System (USCS) Chart

Major Divisions			Group Symbols	Typical Names
Coarse Grained Soils More than 50% retained on No. 200 sieve*	Gravels 50% of more of coarse fraction retained on No.4 sieve	Clean gravels	GW	Well-graded gravel and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravel and gravel-sand mixtures, little or no fines
		Gravels with fines	GM	Silty gravel, gravel-sand-silt mixtures
			GC	Clayey gravel, gravel-sand-clay mixtures
	Sands more than 50% of coarse fraction passes No.4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained soils 50% or more passes No. 200 sieve*	Silts and clays liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and clays liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
			PT	Peat, muck and other highly organic soils

- Based on the material passing a 3-in. (75mm) sieve

Source : The Annual Book of ASTM Standards, 1983

<u>USCS Examples</u>

In practice, soils consist of one or more particle sizes. The soil name "gravel" does not necessarily mean that all the particles present in the soil are of gravel size, but in addition, there may be some of other particle sizes too. The figure below gives some guidelines on naming soils as per the percentage fraction of different sizes present in the soils.

Table No. 2.7: Geotechnical Denomination of Sand and Gravel

Main Property	Denomination of Main Fraction	Denomination of Secondary Fraction: proportion	Denomination of Secondary Fraction: Fraction	Denomination of a Possible Third Fraction
Clean	Gravel +	Little (3 – 15 %)	Sand +	Boulders
Clayey		Much (16 – 30%)		Stones
Silty	Sand +	Very much (31 – 49%)	Gravel +	Organic peat

Table No. 2. 8: Geotechnical Denomination of Soil and Clay

Main Property	Denomination of Main Fraction	Plasticity	Denomination of Secondary Fraction	
			Proportion	**Fraction**
				Boulders
Clayey	Silt +	Null	Low %	Stones
		Low	Medium %	Gravel
Silty	Clay +	Medium	High %	Sand
		High		Peat

Table No. 2.9: Soil Names

Soil Name	Content of Secondary Fraction
Slightly sandy GRAVEL Sandy GRAVEL Very sandy GRAVEL GRAVEL / SAND Very gravelly SAND Gravelly SAND Slightly gravelly SAND	Up to 5% sand 5%-20% sand over 20% sand about equal proportions over 20% gravel 5%-20% gravel up to 5% gravel
Slightly silty SAND (or GRAVEL) Silty SAND (or GRAVEL) Very silty SAND (or GRAVEL)	Up to 5 % silt 5 % - 15 % silt 15 % -35 % silt
Slightly clayey SAND (or GRAVEL) Clayey SAND (or GRAVEL) Very clayey SAND (or GRAVEL)	Up to 5 % clay 5 % -15 % clay 15 % -35 % clay
Sandy SILT (or CLAY) Gravelly SILT (or CLAY)	35 % -65 % sand 35 % -65 % gravel

In this classification system, the field identification of soil is also included. The recommended procedure for field identification is detailed below.

1) Spread the sample on a flat surface, and based on observation, classify the soil as coarse-grained if 50% of the particles are visible to the naked eye, otherwise classify the soil as fine-grained.
2) If coarse-grained, classify as gravel if more than 50% of the coarse fraction is of gravel size (larger than 4.75 mm), otherwise classify as sand. If fine-grained, see (6) below.
3) If gravel or sand, classify as "clean" if it contains little or no fines, otherwise classify it as “with appreciable fines”.
4) Classify “clean gravel or sand” as either well-graded or poorly graded.
5) Classify “gravel or sand” with appreciable fines according to the nature of fines as in (6) below.
6) For fine-grained soil or fine fraction of coarse-grained soil, classify on the basis of behavior in the dilatancy, dry strength and toughness.
7) Highly organic soils are identified by their color, odor, spongy feel and fibrous texture and are classified as peat.

2.2.1.4 Properties of Soil

Index and Strength Properties of Soil

The index properties are of great value to a civil engineer in that, on one hand, they provide the means for the correlation of construction experience, and on the other hand, they form a basis of information of the correctness of the field identification of a given material. If the material has been improperly identified, the index properties indicate the errors and lead to correct classification.

Index properties may be divided into two general types, namely – Soil Grain Properties and Soil Aggregate Properties. The soil grain properties are the properties of the individual particles of which the soil is composed and are independent of the manner of soil formation. These properties can be determined from distributed samples.

Soil aggregate properties, on the other hand, depend on the structure and the arrangement of the particles in the soil mass. Whereas the soil grain properties are commonly used for soil identification and classification, the soil aggregate properties have a greater influence on the engineering behavior of soil mass. The engineering behavior of a soil mass depends on its strength, compressibility and permeability characteristics.

The most important aggregate property of coarse-grained soil is its relative density while that of fine-grained soil is its consistency.

Soil Grain Properties

The most important grain properties of coarse-grained soils are particle size, grain shape and mineralogical composition. The particle size distribution is determined by performing a mechanical analysis in which the soil is passed through a set of sieves having square openings. The finest sieve commonly used in practice is 75-micron, i.e., a sieve with 0.075 mm wide openings. This sieve size demarcates the boundary between coarse-grained and fine-grained soils.

For soils having grain size smaller than 0.075 mm, sieve analysis is not practicable. For soil fractions finer than 0.075 mm, particle size distribution is determined by sedimentation analysis (hydrometer analysis).

GRAIN SIZE

The grain size of coarse-grained soil is determined by sieve analysis. The determination is described below.

Sieve Analysis

About 500 gms of air-dried soil is taken. The soil is passed through a set of sieves arranged in such a way that the sieve with the largest opening is at the top and the sieve with the smallest opening is at the bottom (Figure. 2.1). The set of sieves is shown below.

The soil is shaken for about 10 minutes in a sieve shaker. The soil retained on each sieve is weighed. Then the percentage retained on each sieve is computed. Finally the percentages of particles finer than the sieve sizes used in the analysis are determined.

If the soil used in the analysis contains an appreciable quantity of particles finer than 0.074 mm, the proportion of materials smaller than 0.074 mm size needs a hydrometer analysis for the determination of grain size distribution of the next pore. The opening of a 0.074 mm sieve is so small that sometimes even particles smaller than this size may not pass through the sieve. In such a case, for better results a wet sieve analysis is carried out where the material is washed through a 75-micron sieve. The material retained on the sieve is dried and then further analyzed by sieving it through set sieves as explained earlier.

The result of grain size analysis is presented in the form of a curve plotted in semi-log scale. The percentage finer than certain value is plotted as ordinate to a natural scale while sieve size is plotted as abscissa to a logarithmic scale. A plot of this type has the advantage that materials of equal uniformity are represented by curves of identical shapes whether the soil is coarse-grained or fine-grained. Further, a logarithmic scale is more convenient to plot where grain sizes vary over a wide range. Typical curves of grain size distribution are shown in Figure 2.2.

A grain size distribution curve is also a measure of the uniformity of soils. In a grain size distribution plot, uniform soils are represented by nearly vertical curves. While S-shaped curves are the characteristics of well-graded soils.

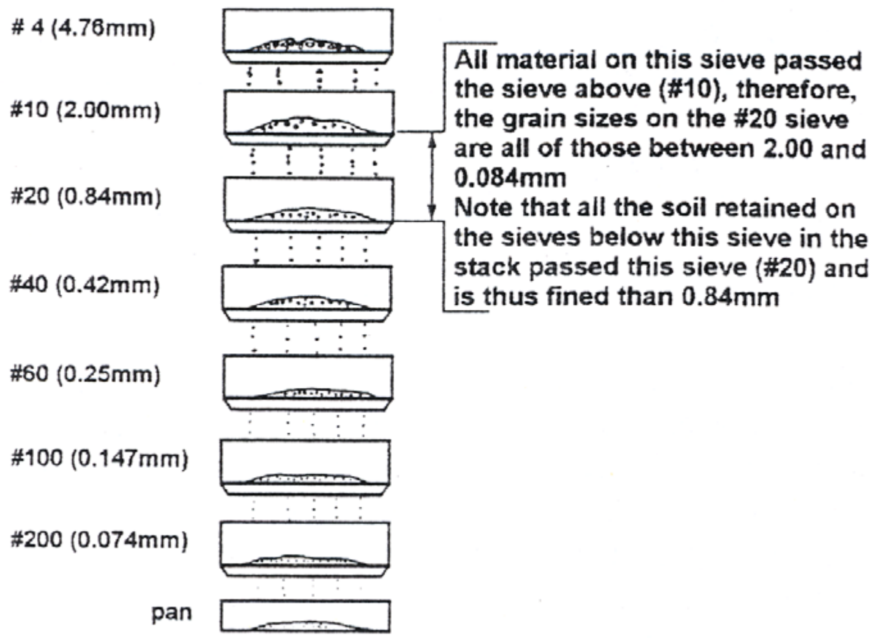


Figure 2.1: Sieve Analysis Set Up

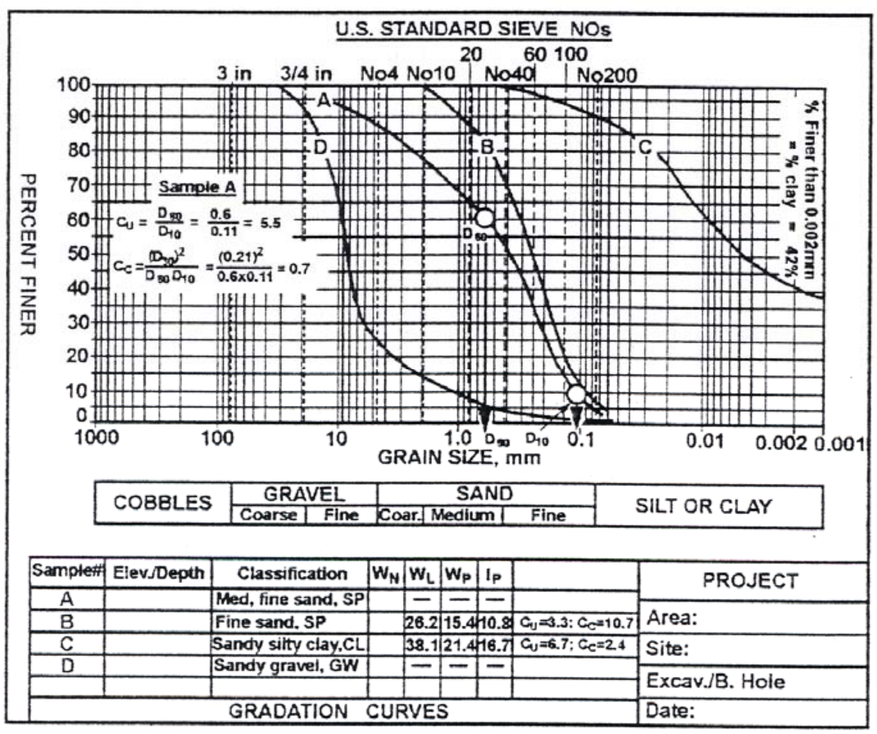


Figure 2. 2: Particle Size Distribution Curve

Co-efficient of Uniformity

The degree of uniformity of a soil is expressed by a number called co-efficient of uniformity. This is defined as the ratio of the diameter of the particle, which has 60 percent of the particles finer than a certain size to the diameter of the particles, which has 10 percent of the particles finer than a certain size.

$$c_u = \frac{D_{60}}{D_{10}}$$

Where,

C_U = Co-efficient of uniformity.

D_{60} = Diameter of particles corresponding to 60% fines.

D_{10} = Diameter of particles corresponding to 10% fines.

Particle size D_{10} is also known as the "effective size".

Co-efficient of Curvature

This term is also used to determine whether a soil is uniform or well graded. This is expressed as:

$$C_c = \frac{D_{30}^{\ 2}}{D_{10} \times D_{60}}$$

Where,

C_c = Co-efficient of curvature.

D_{30} = Diameter corresponding to particles finer than 30 percent.

In well-graded gravel, C_U is greater than 4, and C_c is between 1 and 3. In well-graded sand, C_U is greater than 6, and C_c is between 1 and 3.

GRAIN SHAPE

The shape of the grains is an important property of a soil. It cannot be given a numerical value. In coarse-grained soil, the grain shape is bulky and it is described as angular, sub-angular, sub-rounded, rounded or well-rounded. In fine-grained soil, the soil grain is mostly plate-shaped, and in a few instances, needle shaped. These shapes are found in the Montmorillonite, Illite and Kaolinite group of minerals that constitute clays. The shape of the grains of coarse-grained soils is shown in Figure 2.3.

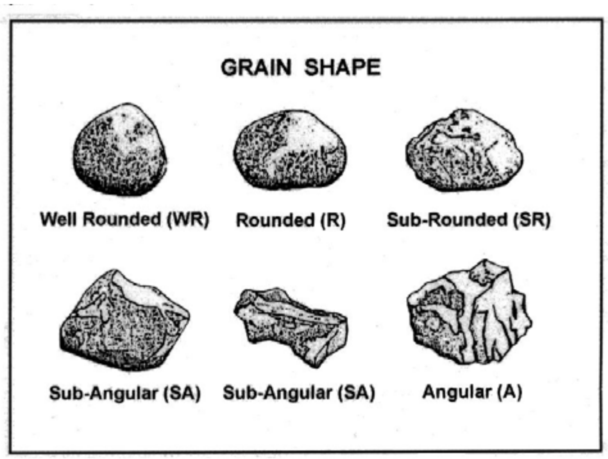


Figure 2.3: Grain Shape of Coarse-Grained Soils

SOIL AGGREGATE PROPERTIES

The important soil aggregate properties are relative density, consistency, unconfined compressive strength, thixotropy, sensitivity and activity.

Relative Density

The most important soil aggregate property of coarse-grained soil is the relative density. The term relative density is used to describe the relative compactness of non-cohesive soil and is generally described as very loose, loose, medium, dense and very dense. In fact, relative density is a function of void ratio and is expressed as:

$$D = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

Where, e_{max} = Maximum void ratio.

e_{min} = Minimum void ratio.

e = Void ratio of a given soil mass.

When soil is very loose, $e = e_{max}$ and $D = 0$.

When soil is very dense, $e = e_{min}$ and $D = 100\%$

Thus soils can have any value of D between 0-100. Depending upon the relative density, soils are generally divided into five categories and are shown below in Table No. 2.10.

Table No. 2.10: Soil States Based On Relative Density

State of Compactness	Very loose	Loose	Medium	Dense	Very Dense
Relative Density, Dr %	< 15	15 - 35	35 - 65	65 – 85	> 85

CONSISTENCY OF SOILS

Consistency is the most important soil aggregate property of fine-grained soils. The physical state of fine-grained soil with a particular water content is known as its consistency. This term denotes the degree of firmness of a soil. The soil consistency is greatly influenced by the presence of clay minerals or organic materials. Qualitatively, soil consistency is expressed as very soft, soft, firm, hard and very hard. Quantitatively, it is expressed in terms of unconfined compressive strength as shown in Table No. 2.11 below:

Table No. 2.11: Consistency of Clay

Consistency	Unconfined Compressive Strength, q_u KN / m^2
Very soft	< 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very stiff	200 – 400
Hard	> 400

Depending upon the water content, a soil can have various states of consistency, i.e., liquid state, plastic state, semi-solid state and solid state. The limiting water content at which a soil passes from one state of consistency to another state is called consistency limit. Consistency limits were first determined by an agronomist, A. Atterberg in 1911, and therefore these limits are also known as

Atterberg's Limits. The limits are somewhat empirical in nature but are of great use in the investigation of plasticity characteristics of clays. The limits of consistency differ from soil to soil and depend on the interaction between the clay mineral particles.

If a large quantity of water is mixed with fine-grained soil, the soil will turn into a liquid state. The soil will not be able to offer any resistance to deformation, i.e., the shear strength of soil will be equal to zero. On gradual reduction of water content and thorough kneading, the soil sample starts offering some resistance to deformation and becomes plastic. The limit at which the suspension passes from zero shear strength to infinitesimal strength is the true liquid limit. However, this point cannot be determined truly. The liquid limit is arbitrarily taken as the moisture content at which the soils have small measurable shear strength.

At a moisture content lower than the liquid limit, the soil is in a plastic state. At plastic state, the soil can be put into any desired shape. On further reduction of the water content, the soil loses its plasticity. The limiting water content above, when the soil will be in a plastic state, is called "the plastic limit". Below the plastic limit, the soil displays the property of a semi-solid. If the water content is further reduced, a state is reached where further reduction in the water content does not produce a reduction in the volume of the soil. The water content corresponding to this state is called the shrinkage limit. Below the shrinkage limit, the volume of the soil becomes constant, the soil becomes partially saturated and the pore water is partly replaced by air. Still further reduction in water content results in dry soil, which is stiff to very stiff. A diagrammatic representation of the different states and limits of soil consistency is shown in Figure 3.4 below.

Plasticity Index

This is defined as the range of water content over which the soil is in a plastic state and called the plasticity index, Ip, which is expressed as:

$$I_P = W_L - W_P$$

Where,

W_L = Liquid limit and W_P = Plastic limit

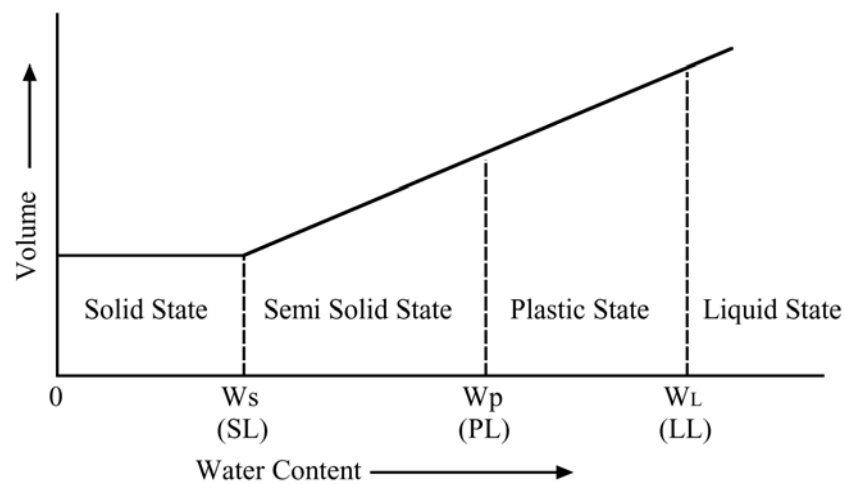


Figure 2.4: States of Consistency limits

DETERMINATION OF CONSISTENCY LIMITS

Liquid Limit

The test procedure for the determination of liquid limit is detailed in IS 2720 (Part V) 1965. The soil sample is dried sufficiently to enable it to be crumbled and broken up by means of a mortar and rubber pestle without crushing individual particles. Material passing a .425mm sieve is used in the test. The liquid limit is measured by means of the set up as shown in Figure 2.5 (from a to d).

The apparatus consists of a flat metal cup mounted on an edge pivot. The cup rests on a hard rubber base. A mechanism enables the cup to be lifted to a height of 10 mm and dropped on the base. The soil paste is prepared by mixing some water with the material passing a .425 mm sieve and is put in the cup and leveled horizontally. The soil in the cup is divided into two halves by cutting a groove in the diameter through the pivot of the cup by means of a standard grooving tool as shown in Figure. II-5. The handle of the liquid limit device is rotated at the rate of two revolutions per second. Suddenly the two halves of the soil in the cup gradually start flowing towards the bottom of the cup. The number of blows required to close the bottom of the groove through a distance of 13 mm is recorded. The water content of the soil in the cup is then determined. The test is repeated at least four times and the water content is increased during each test. The number of blows is kept between 50 and 10.

A graph is then plotted between the water content and the logarithm of the number of blows and the best straight-line fitting the plotted points are drawn. For this test, the liquid limit is defined as the water content at which 25 blows are required to close the bottom of the groove over a distance of 13 mm. The typical result of a liquid limit test is given in Figure 3.5.

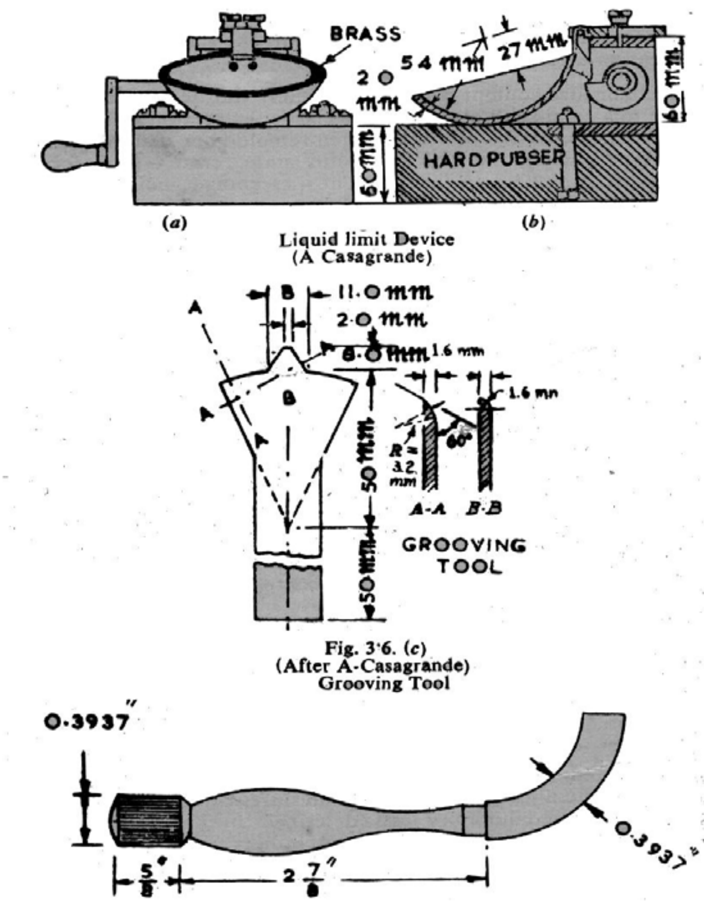


Figure. 2.5 - Liquid Limit Test set Up

Plastic Limit

To determine the plastic limit, dried soil passing through a 0.425mm sieve is mixed with clean water until it becomes sufficiently plastic to be molded into a ball. About 2.5gm of the soil paste is formed into a thread of about 6 mm diameter between the first finger and the thumb of each hand. The thread is then placed on a glass plate and rolled with the tips of the fingers of one hand until its diameter is reduced to about 3mm. Throughout the test, the rolling pressure is kept uniform. The thread is then remolded between the fingers and the procedure is repeated until the thread of the soil shear both longitudinally and laterally when it has been rolled into a thread of 3 mm diameter. The procedure is repeated for three more tests and the percentage of water content of all the crumbled soil is determined as a whole. The water content obtained is the plastic limit of the soil. If the plastic limit cannot be determined for a given soil or if the plastic limit is equal to or greater than the liquid limit, the soil is said to be non-plastic (NP).

Shrinkage Limit

The shrinkage limit can be found from both undisturbed as well as disturbed soil samples. A remolded soil sample for the determination of shrinkage limit is prepared with moisture content above the liquid limit and then it is allowed to dry in an oven. A phase diagram showing the determination of the shrinkage limit of a soil is shown in Figure 3.6 (a), (b), and (c).

The soil is fully saturated up to the shrinkage limit. Figure II-6 (a) shows a soil in plastic state. Its volume in this state is V_1 and its corresponding weight is W_1 out of which W_s is the weight of the soil solids. The volume V_1 of the soil can be found out from the known volume of the shrinkage dish.

The soil in the shrinkage dish is dried slowly so as not to form any cracks. The dry weight W_s of the soil is found out. The volume V_2 of the dry soil pat is found by immersing the pat in a known volume of mercury. The mercury displaced and spilled out of the shrinkage dish is weighed and divided by the unit weight of mercury, which gives the volume V_2 of the soil pat at the shrinkage limit. It is required to find out the weight of the water W_w at the shrinkage limit.

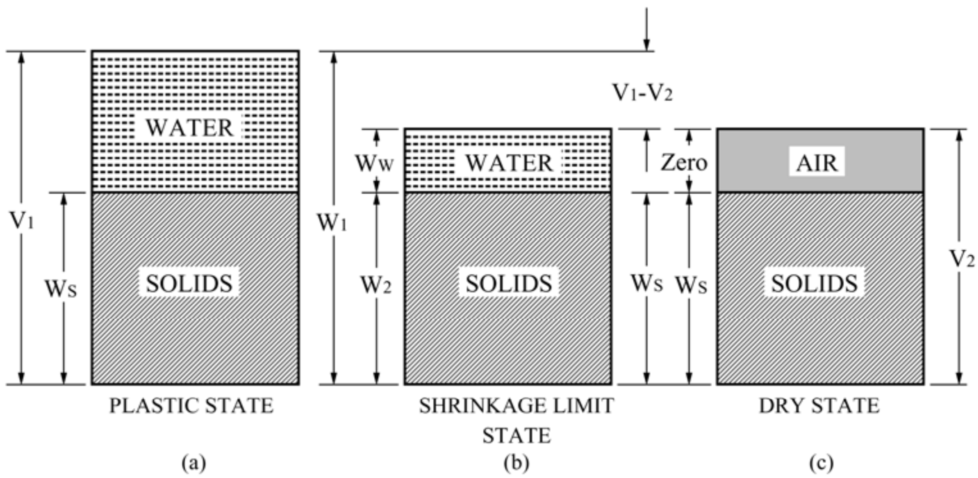


Figure 2.6: a), b) and c) phase diagram showing the determination of the shrinkage limit of a soil sample

Wt. of water evaporated above shrinkage limit $= (V_1 - V_2)\gamma_w$

Wt. of water W_w at shrinkage limit $= (W_1 - W_2) - \gamma_W (V_1 - V_2)$

$\therefore$ Shrinkage limit, $W_w = \frac{W_w}{W_s} \times 100$

$$= \frac{(W_1 - W_s) - \gamma_W (V_1 - V)_2}{W_s} \times 100$$

Unconfined Compressive Strength

It is defined as the load per unit area at which a cylindrical soil specimen of height to diameter ratio of 2 will fail in a simple compression test. The unconfined compressive strength is a measure of the soil consistency and is presented in Table 2.11.

Sensitivity

It is a measure of the effect of remolding on the strength of the soil without any change in its water content. Numerically, the sensitivity S_t is expressed as:

$$S_t = \frac{\text{Unconfined compressive Strength (undisturbed)}}{\text{Unconfined compressive Strength (remolded)}}$$

Depending upon the values of the sensitivity, natural clays are termed as normal, sensitive, extra-sensitive and quick. The degree of sensitivity is given in Table No. 2.12 below.

Table No. 2.12: Sensitivity of Clays

Sensitivity	Classification
1 - 4	Normal
4 - 8	Sensitive
8 - 15	Extra-sensitive
> 15	Quick

Stiff fissured over consolidated clays have sensitivity less than unity.

Thixotropy

It is the property of a material by virtue of which loss of strength due to remolding is regained in a short period of time. The increase in strength is attributed to the rearrangement of the soil and water molecules. The loss of strength due to remolding is due to the destruction of the structural arrangement of the soil particles. Piles driven in clays utilise this property to develop skin friction.

Activity of Clay

It is the ratio of plasticity index to percentage of particles finer than 75 micron. Numerically it is expressed as:

$$A = \frac{I_p}{\%\,\text{finer than } 75}$$

Where,

I_p = Plasticity Index

Based on activity, the classification of clayey soil is presented in Table No. 3.13 below.

Table No. 2.13: Classification of Soil Based on Activity

S. No.	Activity	Classification
1.	< 0.75	Inactive
2.	0.75 - 1.25	Normal
3.	> 1.25	Active

Liquidity Index

The liquidity index I_L is defined as the ratio of difference in natural water content and water content at plastic limit to the plasticity index of soil. Numerically it is expressed as:

$$I_L = \frac{W - W_p}{I_p} \times 100$$

Where, w = Natural water content of soil.

The liquidity index of a soil indicates the nearness of its water content to its liquid limit. For the natural soil with its water content equal to liquid limit, I_L = 100%. This means that if the soil is remolded, it transforms into viscous slurry and behaves as a liquid, i.e., the shear strength is zero. When the natural water content is close to the plastic limit, I_L = 0, it indicates that the soil cannot be remolded. If I_L = 0, the soil will be in a hard state.

GENERAL DISCUSSIONS ON CONSISTENCY LIMITS

The consistency limits are of great practical importance for fine-grained soils. These limits can be correlated to the engineering properties of soil such as strength, permeability and compressibility and provide a rough estimate of the engineering behavior of in-situ soils. Some important information on the basis of soil consistency is presented below:

1) Both the liquid and plastic limits depend on the type and amount of clay present in the soils. The plasticity index mainly depends on the amount of clay. A higher value of the plasticity index indicates a larger percentage of clay in a soil mass.
2) Both the plastic and liquid limits increase with a decrease in particle size. The liquid limit increases at a faster rate than the plastic limit as the amount of clay in the soil increases. As a result, the plasticity index increases at a rapid rate.
3) The plasticity index and the liquid limit enable classification of fine-grained soil. In fact, the main use of consistency limits lies in the classification of soils.
4) Soil with a liquid limit less than 20 is sand.
5) The plastic limit increases with an increase in the organic content in the soil. However, an increase in the organic content increases the liquid limit only marginally. Thus organic clays exhibit high plastic limit values.
6) The liquid limit of a soil is an indicator of the compressibility of soils. For normally consolidated clay, the compression index C_c is generally expressed in terms of the liquid limit as:

$$C_c = 0.009\,(W_L - 20) \quad \text{Where,} \quad W_L = \text{Liquid limit.}$$

7) At equal plasticity index, as the liquid limit is increased, the dry strength and toughness decrease and compressibility and permeability increase.
8) At equal liquid limit, with an increase in the plasticity index, the dry strength and toughness increases, the permeability decreases but the compressibility remains the same.

2.2.1.5 Field Identification of Soil

Procedure

The identification of soils in the field without carrying out laboratory tests is known as field identification of soils. The principal soil types in civil engineering are gravel, sand, silt and clay. These types do not occur in nature freely and always remain as a mixture of two or more varieties. In the mixture, if sand is present as a major component and silt as a minor component, then the soil is called Silty Sand. Thus the purpose of field identification is to classify the soils in the field roughly.

As discussed earlier, soils are broadly classified as coarse-grained and fine-grained soils. Thus sand and gravel, pebbles, cobbles and boulders fall under the category of coarse-grained soils. Silt and clay fall in the category of fine-grained soils. The main distinguishing features between these soils are whether they are visible to the naked eye or not. The smallest particle of coarse-grained soil which can be seen by the naked eye is sand. Thus, if the particles are visible to the naked eye, the soil is coarse-grained. If they are not visible to the naked eye, they are fine-grained, i.e., silt or clay.

Field Identification Procedure

Boulders, Cobbles and Pebbles

Coarse-grained soils are classified based on their grain size. The size of a boulder is greater than 250 mm. A particle sized between 60 mm and 250 mm is cobble. The size of a pebble is between 4.75 and 60 mm.

Gravel versus Sand

Any individual particle of soil larger than 4.75 mm is gravel. A particle between 4.75 mm and 0.075 mm is sand. Roughly, gravel is larger than a children's marble. If the individual soil particles are smaller than a children's marble and is visible to the naked eye, the soil is sand.

Field identification of gravel and sand should also include identification of mineralogical composition and the shape if possible. The shape may be described as stated in the earlier session. Figure 2.7 provides the ratio of coarse grains and the shape of the grains. Figure 2.7 shows a table to identify the soil in the field.

Sand versus Silt

Dispersion Test - It would be difficult to distinguish sand and silt by visual examination. To distinguish them a dispersion test is carried out. In this test, a spoonful of the sample is poured into a small measuring cylinder and shaken uniformly. If the material is sand, it will settle down within a minute. If it is silt, it will take anywhere from 15 minutes to one hour to settle down. In both cases, nothing will be left in suspension after the lapse of the respective timings.

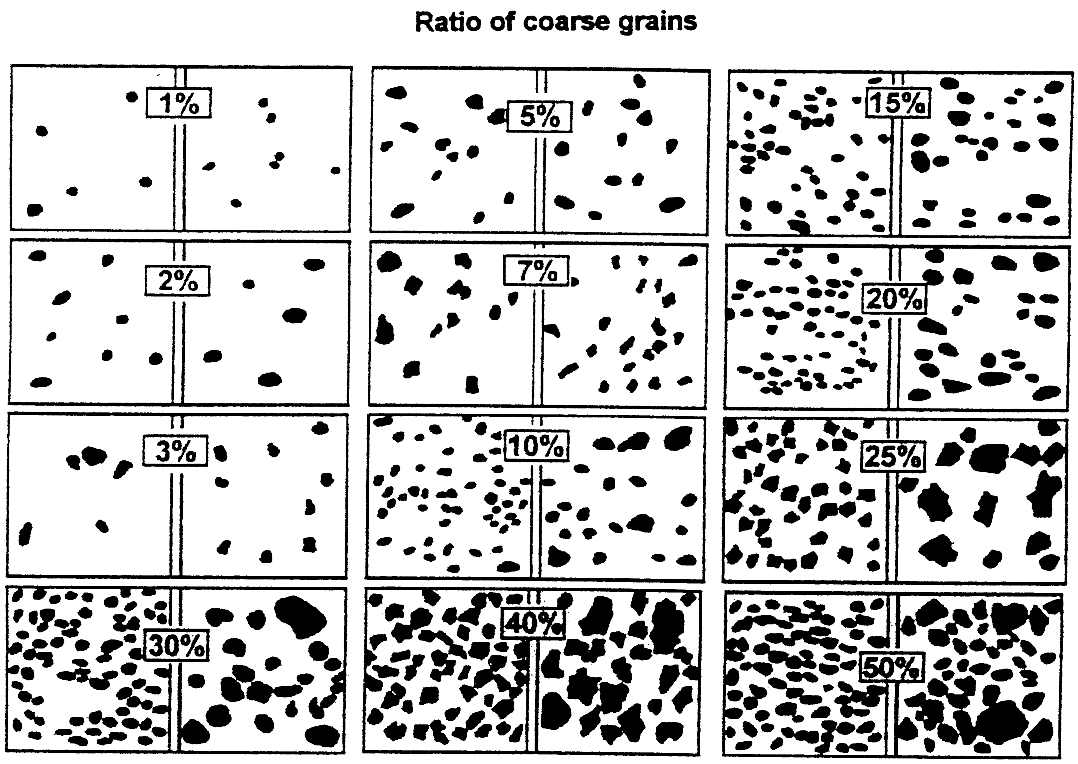


Figure 2.7: Ratio of Coarse Grains and Shape of the Grains

Silt versus Clay

To distinguish between silt and clay, the following test has to be conducted.

Shaking Test - In this test, a pat of material is prepared and placed in the open palm of one hand and shaken horizontally by striking vigorously against the other hand several times as shown in Figure 2.8. If silt is present, due to its high permeability, water will appear on the surface of the pat. The appearance of water on the surface of the pat will give a shining surface. When the sample is squeezed between the fingers, the water and gloss disappear from the surface and get absorbed in the soil. If it is clay, due to its lower permeability, on shaking, water will not come to the surface and the surface will be dull.

If silt and clay are to be distinguished, their relative presence can be judged by observing the time taken by the water to come to the surface. If the reaction is rapid, the material is predominantly silt. If the reaction is slow, the quantity of silt is relatively less. If water does not come to the surface and the surface is dull, then it means that the sample is clay and there is no silt in it.

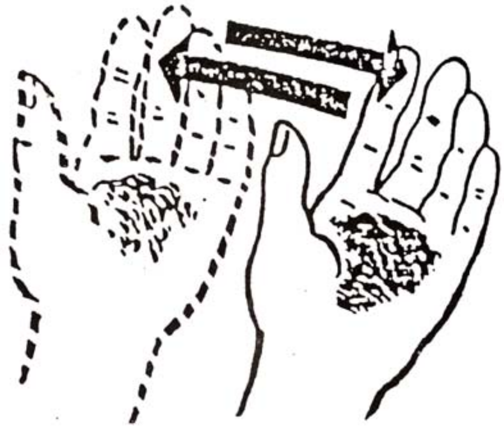


Figure 2.8: Shaking Test

Dispersion test - It would be difficult to distinguish between clay and silt by visual examination. To distinguish them, a dispersion test is carried out. In this test, a spoonful of the sample is poured into a small measuring cylinder and shaken uniformly. If the material is silt, it will take anywhere from 15 minutes to one hour to settle down. In case the soil is predominantly clay, it will form a suspension which will remain as such for hours and may be for days, unless the material is allowed to flocculate by adding some flocculating chemicals.

Ribbon Test - A soil which is predominantly clay can be rolled (Figure 2.9) in the form of a thread of about 3 mm in diameter without disintegrating. A thread of this size made out of clay and about 30 cm long can support its own weight, when held in the hand at the two edges. In the case of pure silt, it would not be possible to make a thread of this size without crumbling or disintegration.

A. Method of rolling thread.

B. Thread of soil above plastic limit.

C. Crumbling thread as plastic limit is reached.

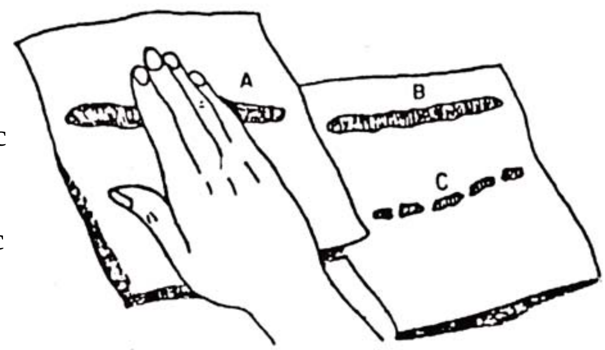


Figure 2.9: Ribbon Test

Wet and Manipulated Strength Test - This test helps to distinguish the predominant soil characteristics, that is whether it is clayey, silty or sandy. In this test, a small quantity of soil specimen is taken in the hand and moistened. The specimen is worked between the fingers. While doing this, if a soapy touch is felt, the soil is clayey. If roughness is experienced, the soil is sandy. When the specimen is squeezed between the fingers and if water comes out, the soil is silty. If the soil is clay, it sticks to the fingers and dries slowly. On the other hand, if the soil is silt, it dries quickly, and when dry, it can be easily dusted off the fingers leaving only a stain.

Crumbling Test. The test permits estimation of the cohesiveness of the soil and therefore its clay content. It can be done by crumbling test as shown in the Figure 2.10.

Method of Crumbling Soil between fingers.

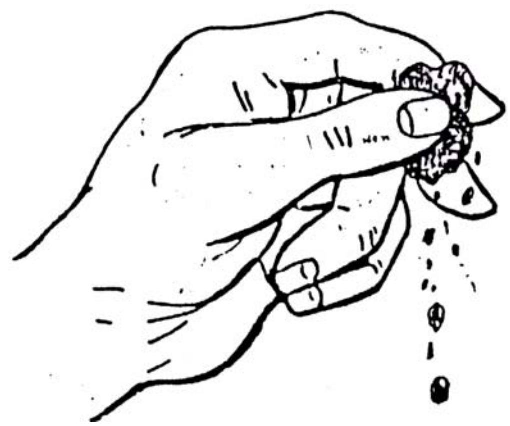


Figure 2.10: Crumbling Test

After removing the particles larger than 0.425 mm, a pat of the soil is molded to the consistency of putty after adding some water. The pat is then dried completely in the sun, in the air or, if available in an oven. Breaking and crumbling between the fingers, the dry strength increases with increasing plasticity, i.e, its clay content.

A *low dry strength* indicates silt, rock flour or silty sand/sandy silt. The sand, however, feels gritty when powdered. The dry pat can be powdered with slight finger pressure. A *medium dry strength* indicates low to medium plastic inorganic clay. Considerable finger pressure is required to powder the sample. A *high dry strength* indicates highly plastic, inorganic clay. The dried sample can be broken but cannot be powdered by finger pressure.

Remark: Cohesion or high dry strength may be due to the presence of cementing material such as calcium carbonate or iron oxide.

Field Test for Estimating ϕ and γ of Fine Non-Cohesive Soils and Soil Matrix

After the completion of the field test and classifying the soil type, it is necessary to estimate its angle of internal friction Φ and unit weight γ (dry, wet, and saturated). For this purpose, it is necessary to have an estimate of the relative density of the soil or the matrix (in case of coarse soil). Table No. 2.14 gives the guidelines to estimate the relative density of soils, which is valid only for coarse-grained soils.

Table No. 2.14: Relative Density Estimate by Penetration Test

Term	Relative density	Penetration field test
Very loose to loose	0 -50%	Easily penetrated with ½ in. reinforcing rod push by hand
Moderately dense	50-70%	Easily penetrated with ½ in. reinforcing rod driven with 5 lb hammer
Dense	.70-90%	Penetrated a foot with ½ in. reinforcing rod driven with 5 lb hammer
Very dense	90-100%	Penetrated a few inches with ½ in. reinforcing rod driven with 5 lb hammer

Once the relative density is estimated by making the test, Figure 2.11 is used to estimate the values of Φ and γ.

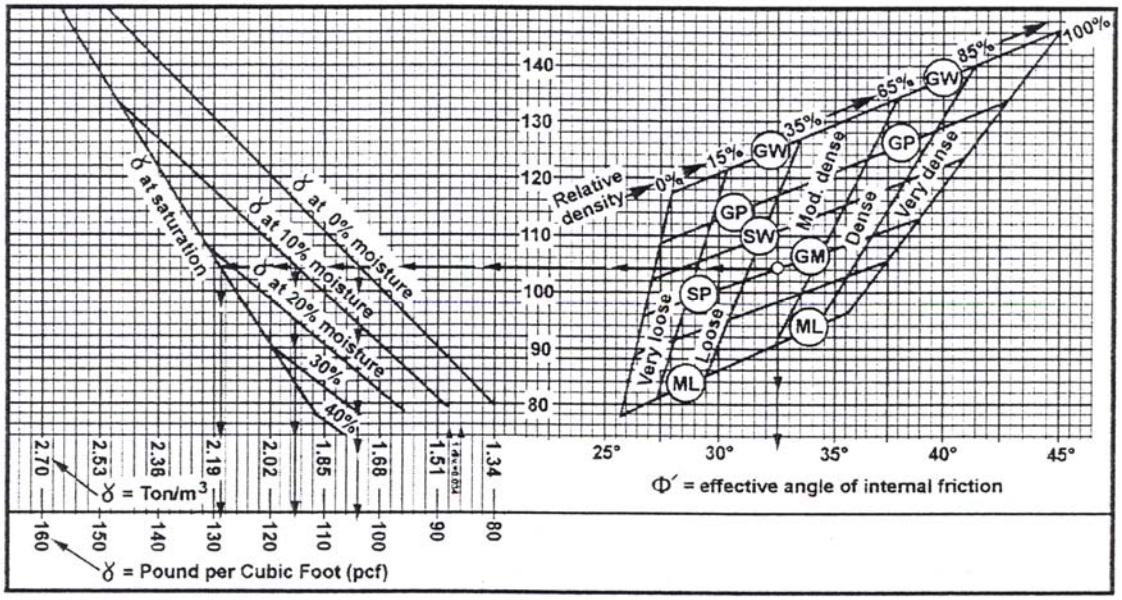


Figure 2.11: Relative Density Estimates for the Values of Φ and γ

2.2.2 Rock Investigation

Rocks are the naturally occurring mineral aggregates. They can be identified on the basis of their mineral constituents. The rocks in the hand specimen may not always be the best representatives of the site conditions. However, it is important to examine the rock closely in the sample and then get the general picture of the site conditions. The geological classification of rocks gives an insight into the processes of rock formation and changes, which is different from the engineering classification of rock.

2.2.2.1 The Rock and the Rock Structures

A large volume of rock interrupted by discontinuities is called Rock Mass. Rock Mass is also defined as;

Rock Mass = Rock Material + Discontinuities

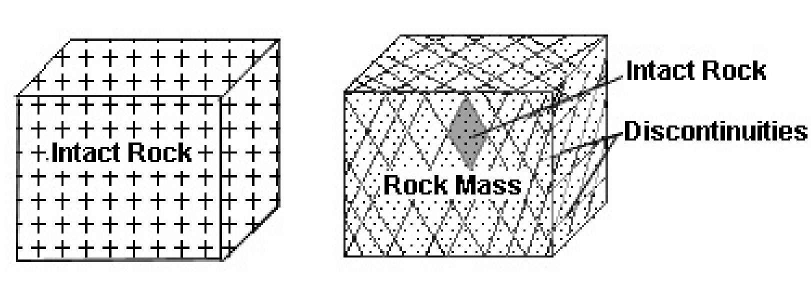


Figure 2.12: Block Diagram of Intact Rock and Rock Mass

An intact rock is a rock sample without any visible discontinuities such as bedding plains, foliations, joints, faults and fractures. The intact rock is also called Rock Material. Which is the ideal condition in practice for disaster managers? The resultant rock mass is a discontinued aggregation of blocks, plates or irregular geometric forms having significantly different physical properties compared with the intact rock sample of the same rock mass. Presence of discontinuities is the primary controlling factor of the rock mass strength and deformability. The stability and deformability of a rocky terrain is dependent on the rock mass.

Evaluation of the engineering properties of a rock mass includes knowledge of the intact rock properties, occurrence and nature of discontinuities and the degree as well as extent of chemical weathering.

Intact Rock

Intact rock materials are commonly described by the geological rock name, mineralogy, texture, degree and the kind of cementation and weathering. The geological properties of the rock material play a decisive role in showing the following engineering properties of the intact rock.

Intact Rock Strength

Strength is the fundamental quantitative engineering property of rock material of concern to disaster managers. Rock strength is the amount of applied stress at rock failure or rupture. Depending upon the applied compressive shear or tensile stress, the strength is also compressive, shear or tensile.

Weathering

Rocks are affected by weathering. Weathering is defined as, "The physical and chemical alteration of rock by the action of heat, water and air". Note that high temperature and high water content increase the rate of weathering.

A weathered rock sample will show some or all of the following features:

- Softness (i.e., minerals can be rubbed off by hand);
- Discoloration;
- Loosening of grains;
- Intact white mica;
- Intact quartz.

The relative order of susceptibility to chemical alteration in the common mineral groups is as follows:

- Dark minerals | Least resistant to alteration
- Light minerals |
- White mica |
- Quartz ↓ Most resistant to alteration

In soil, weathering has preceded much further and the following features may be observed:

- Particles are much smaller;
- Clay minerals (fines) are present - these are new minerals derived from the weathering products of rock;
- Quartz and white mica remain, as they are most resistant to weathering.

The effect of weathering is to soften and weaken the rock.

The mechanical disintegration as well as chemical decomposition of rock under the influence of atmospheric conditions is called weathering. Reduction in compressive strength is the most important geotechnical factor caused by chemical weathering or alteration of intact rock. Long-term weathering has an influence on the physical state or condition of the rock at a given site. Short-term weathering processes may create problems during construction by exposing the rock to the surface. This susceptibility of rock to short-term weathering is called weatherability.

Slaking or breakdown of some clay bearing rock when they are wet is an example of short-term weathering. The Geological Society London, 1970 had proposed a weathering classification with five grades ranging from fresh to completely weathered rock (Table 2.15).

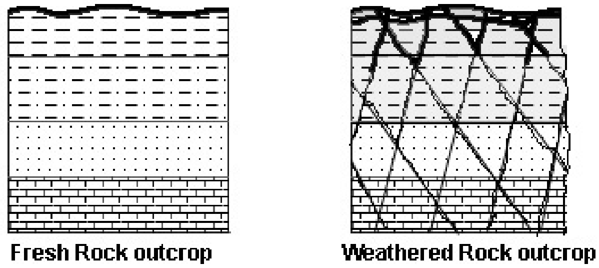


Figure. 2.13: Fresh and Weathered Rock Outcrop

Table No. 2.15: Standard Terminology for Description of Weathering of Rock Mass

Weathering description	Grade number (Anon, 1977)	Weathering phase (Lemppet al. 1985)	Description
Fresh	IA	W0	No visible sign of weathering
Faintly weathered	IB		Weathering limited to the surface of major discontinuities
Slightly weathered	II	W1	Weathering penetrates through most discontinuities, but only slight weathering of the rock material
Moderately weathered	III	W2	Less than half the rock decomposed to soil. Discolored, perhaps fresh rock, present as a continuous framework.
Highly weathered	IV	W3	More than half the rock weathered to soil. Discolored, perhaps even some fresh rock present as discontinuous framework.
Completely weathered	V	W4	All rock weathered to soil. Structure and texture largely intact.
Residual soil	VI	W5	All rock weathered to soil. All texture and structure destroyed. No significant transportation of soil visible.

Table No. 2.16: Weathering classification and rating based on the recognition of engineering significance (Price, 1993)

Class	Rating	Descriptive term	Engineering significance
A	140	Effectively weathered	No influence of weathering so the engineering problems are related to the intact rock properties and discontinuity properties
B	100	Significantly weathered	Reduction in the strength along the discontinuity planes may increase instability phenomena at the surface as well as at tunnel walls and in the foundations.
C	50	Severely weathered	Irregular bearing capacity, major slope stability by release of core-stones, slope stability approaches the stability of residual soil, poor tunneling conditions
D1	20-0	Geotechnical soil without relict discontinuities	The weathered material behaves as geotechnical soil, so that the engineering works should be designed on the basis of soil parameters.
D2	-20	Geotechnical soil without relict discontinuities	The weathered material behaves as geotechnical soil, so that the engineering works should be designed on the basis of soil parameters, but potential sliding planes of relict discontinuities may affect the conditions negatively.

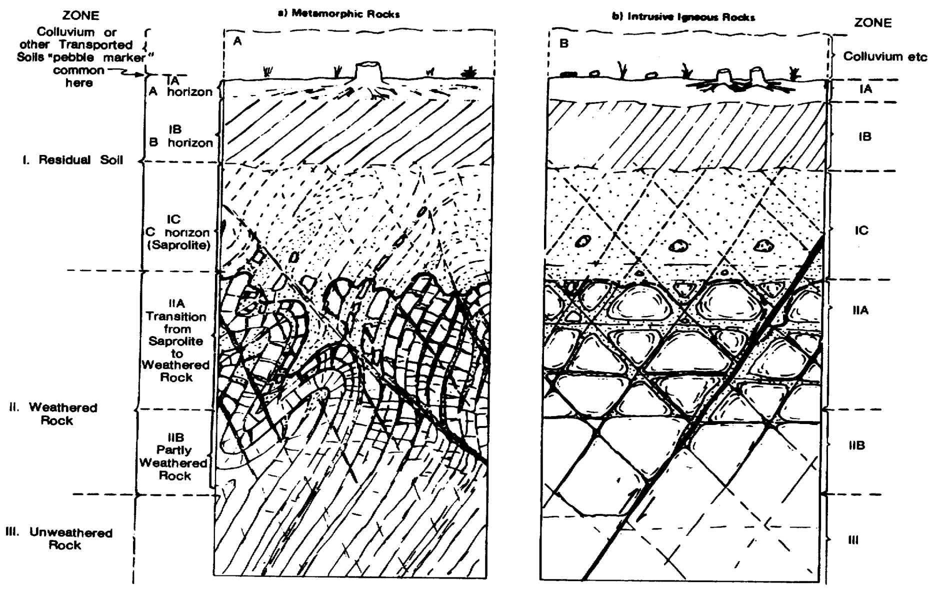


Figure 4.40 Typical weathering profiles for (*a*) metamorphic and (*b*) intrusive igneous rocks. (Reprinted with permission from Proc. 4th Pan American Conference on Soil Mechanics and Foundation Engineering, D. U. Deere and F. D. Patton, Slope Stability in Residual Soils, 1971, Fig. 1, p. 90, Am. Soc. Civil Eng.)

Discontinuities

These are the distinct breaks in the physical continuity of the rock. These include bedding surfaces, joints, faults and well developed metamorphic foliation. Presence of the discontinuities results in the rock mass turning into a discontinuous aggregation of blocks, plates or irregular geometric forms. They have significantly different physical properties compared to intact rock. There are generally three to four sets of discontinuities in most of the rock outcrops.

Shear strength of the rock mass mostly depends on the characteristics of the discontinuities. The most consistently measured joint properties or factors are:

* Orientation
* Spacing
* Continuity
* Surface characteristics
* The separation of discontinuity surface
* Accompanying thickness and nature of filling material (if present)

Orientation

The attitude of the discontinuity in the space described by the dip direction (azimuth) and the dip of the line of the steepest declination in the plane of the discontinuity is called its orientation. Orientation of discontinuities directly influences the rock mass strength and is evident in the failure of rock slopes along one or more discontinuities. Hence, Orientations of the discontinuities are one of the major factors leading to instabilities. Most of the Discontinuities are approximately parallel to each other and constitute what is called a "set". They may be closely or widely spaced. Most rocks contain several sets of discontinuities. Unfavorable orientation of the discontinuities is the main cause of failure of rock slopes.

Note that rocks may split:

- along widely-spaced planes, following the grain of the rock;
- along many closely-spaced planes, following the grain of the rock;
- across the grain of the rock at regular spacing and orientation. These fractures are called joints.

Friction along the interfaces between the blocks governs the shear strength of the rock. Shear strength is reduced when contact along the interfaces is lost. Rock strength is related to the number and weakness of fractures. Strong rocks have fewer fractures or closed and cemented fractures.

Open-jointed rocks are very weak because:

- water movement and weathering take place preferentially along the joint planes;
- there is loss of frictional resistance along the interfaces.

The degree of weathering of the rock itself also controls rock strength. A highly weathered rock may fail through the rock body rather than along the joints.

Rocks are penetrated by fractures of various spacing and orientation.

Rock strength is due to both:

- spacing and weakness of fractures;
- degree of weathering of the body of the rock.

Fractures are the main cause of failure in rock slopes.

Notice the following features of the rock:

- bedding;
- orientation of structures;
- fracturing and jointing.

The orientation of these planes controls the resistance of the rock to gravitational forces.

Disaster managers should be well familiar with the interrelation between discontinuity orientation and possible major failures.

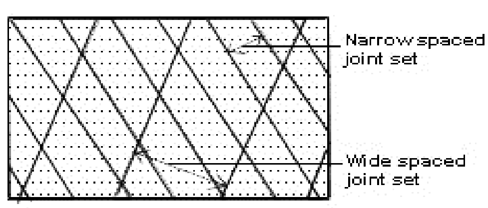
Rock slope may be unstable if:

The discontinuities or intersection of discontinuities cut or day light the slope at less than the natural or manmade slope angle.

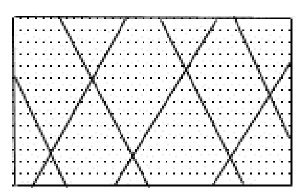
The dip angles of the discontinuities or the plunge angles of the intersection of discontinuities exceed the angle of friction along the discontinuity surface.

Spacing

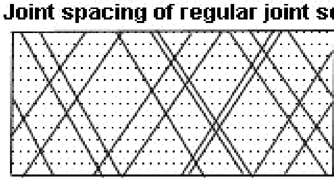
The perpendicular distance between adjacent discontinuities is called their spacing. It normally refers to the mean or modal spacing of a set of joints. The strength as well as the quality of the rock mass decreases on decreasing the spacing of the discontinuities. When they are closely spaced joints, even the strongest intact rock will have very little strength hence the very poor quality. On intersecting and day lighting, discontinuity surfaces are encountered in natural or cut slopes. Closely spaced discontinuity surfaces tend to cause numerous rock falls or reveling of the surfaces, whereas widely spaced discontinuities may tend to cause massive catastrophic block failures. Spacing distance or frequency of discontinuity may vary throughout the rock mass as:



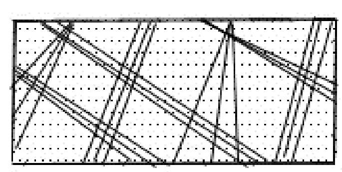
Joint spacing of regular joint sets



Evenly spaced joint sets



Randomly spaced joint sets

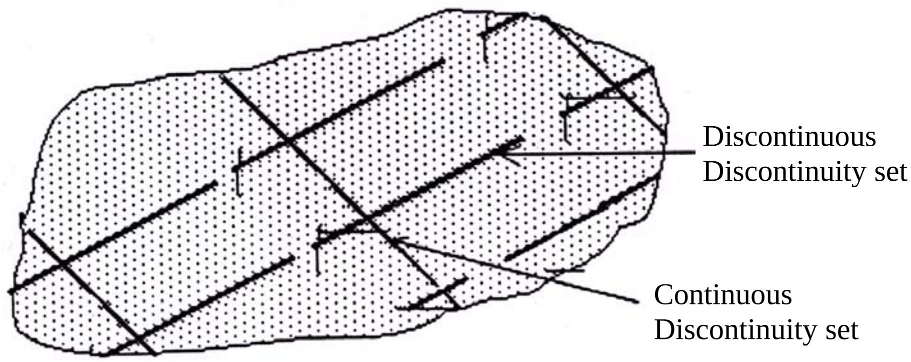


Cluster spaced joint sets

* Even spacing
* Random spacing
* Cluster distributions

The factors contributing to joint spacing are:

* Lithology
* Tectonic stress
* Overburden stress and
* Depth



Continuity

The length of continuity of the discontinuities is generally measured on an exposed rock surface. The continuity, persistence or sizes of discontinuity in a given set decide whether the strength of the intact rock is transferred to the rock mass through the intervening rock or rock bridges. Maximum reduction in rock mass strength is not achieved until the intact rock along the rock bridge fails.

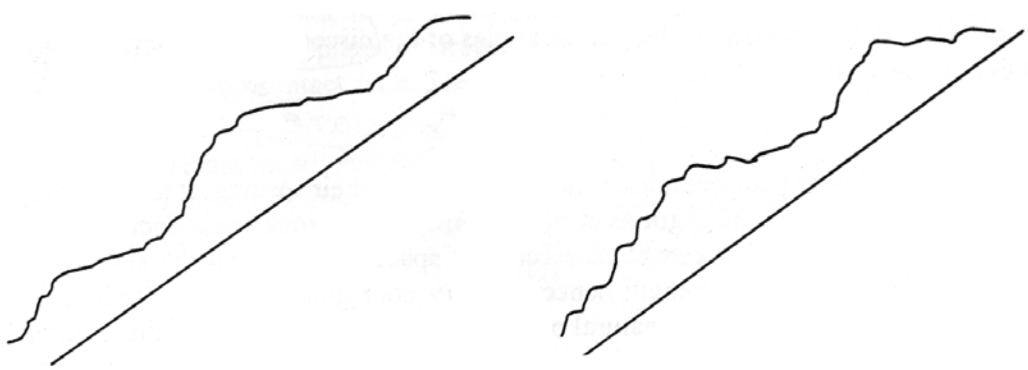


Figure 2.14: Irregularities in the Discontinuities

Surface Characteristics

Following surface characteristics of the discontinuities influence the strength of the rock mass:

- The waviness or undulation of the surface resulting in variation in orientation or attitude along a given discontinuity.
- The smaller scale roughness of the surface resulting in friction between two adjacent blocks.
- Physical property of the material along the discontinuity surface.

Separation and Filling of Discontinuities

The strength of a rock mass also depends on the degree of separation or the width of the space between discontinuity surfaces and the presence of filling material. Such separations are the product of tensile stress during their genesis.

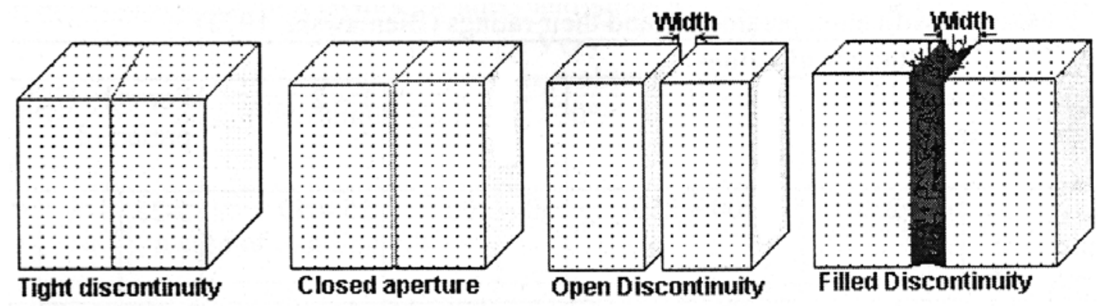


Figure 2.15: Separation and infilling of the discontinuities

Such resultant spaces due to separation may be clear, partly filled or completely filled with materials such as mineralized clay, sand silt, coarse fragment material or mixtures of them resulting from depositional filling, faulting or wall rock weathering. Due to the enormous variety of occurrences, filled discontinuities display a wide range of physical behaviors particularly shears strength, deformability and permeability. Their wide range of behaviors depends on:

- Mineralogy of filling material
- Grading or particle size
- Over consolidation ratio
- Water content and permeability
- Previous shear displacement
- Wall roughness
- Width
- Fracturing or crushing of wall rock

A rock mass having filled discontinuity spaces has generally lower shearing strength except due to mineralized filling. The separation width of a filled discontinuity influences the shear strength along its surface. The wider the separation, the lower is the possibility of friction properties of the discontinuity surface and failure will follow the criteria of the mechanics of the filled material.

Alteration of Discontinuity Wall

Any sort of chemical changes in the atmospheric condition results in alteration along the discontinuity surfaces. Such altered discontinuity surfaces ultimately reduce the shearing strength of the rock mass. Weathering generally affects the walls of the discontinuities more than the interior of the rock blocks. This results in wall strength different than the strength measured in fresh rock. The apparent uniaxial compressive strength can be estimated by the Schmidt hammer test and by the scratch and geological hammer test. The compressive strength of a rock mass comprising the walls of a discontinuity is a very important component for shear strength and deformability, especially when the walls are in direct rock-to-rock contact as in the case of unfilled joints.

Activity of Water along the Discontinuity

A rock mass may be completely dry, damp or wet, ground water may be dripping or flowing through the discontinuities. Dry conditions result in a maximum and flowing conditions result in a minimum shearing strength of the rock mass.

Conclusive remarks

The mechanical behavior of the rock mass is governed by the characteristics of the discontinuities. The characteristics of the discontinuities are studied at the site. Careful studies of the discontinuities have to be carried out at the site during the selection and evaluation of the trail bridge site.

2.2.2.2 Rock Mass Classification

Rock Mass is classified on the basis of the characteristics of the Intact Rock and discontinuities. Various classification systems of the Rock Mass have been developed for various purposes. The Rock Mass Rating (RMR) system is generally used in the Trail Bridge sector. The parameters of the rock mass and their rating for the classification of the Rock Mass using the RMR system is tabulated below in Table No. 2.17.

Table No. 2.17: Rock Mass Classification Parameters and their Ratings (Bieniawski, 1973)

<table>
<tr><td colspan="3">Parameters</td><td colspan="7">Ranges of values</td></tr>
<tr><td rowspan="3">1</td><td rowspan="2">Intact rock strength</td><td>Point load strength index (MPa)</td><td>>10</td><td>4 – 10</td><td>2 – 4</td><td>1 – 2</td><td colspan="3">For <1, UCS is preferred</td></tr>
<tr><td>Uniaxial compressive strength (MPa)</td><td>>250</td><td>100 – 250</td><td>50 – 100</td><td>25 – 50</td><td>5-25</td><td>1-5</td><td><1</td></tr>
<tr><td colspan="2">Rating</td><td>15</td><td>12</td><td>7</td><td>4</td><td>2</td><td>1</td><td>0</td></tr>
<tr><td rowspan="2">2</td><td colspan="2">RQD (%)</td><td>90 – 100</td><td>75 – 90</td><td>50 – 75</td><td>25 – 50</td><td colspan="3"><25</td></tr>
<tr><td colspan="2">Rating</td><td>20</td><td>15</td><td>10</td><td>8</td><td colspan="3">3</td></tr>
<tr><td rowspan="2">3</td><td colspan="2">Discontinuity spacing (m)</td><td>2.0</td><td>0.6 – 2.0</td><td>0.2 – 0.6</td><td>0.06-0.2</td><td colspan="3"><0.06</td></tr>
<tr><td colspan="2">Rating</td><td>20</td><td>15</td><td>10</td><td>8</td><td colspan="3">5</td></tr>
<tr><td rowspan="2">4</td><td colspan="2">Condition of discontinuities</td><td>Very rough surface, not continuous, tight, fresh wall</td><td>Slightly rough surface, low continuous, <1mm opening, slightly weathered wall</td><td>Slightly rough surface, continuous, <1mm opening, highly weathered wall</td><td>Slicken slid surface or Gauge <5mm thick, continuous, 1-5mm opening, highly weathered wall</td><td colspan="3">Soft Gauge >5mm thick, continuous, > 5mm opening, highly weathered wall</td></tr>
<tr><td colspan="2">Rating</td><td>30</td><td>25</td><td>20</td><td>10</td><td colspan="3">0</td></tr>
<tr><td rowspan="4">5</td><td rowspan="3">Ground water</td><td>Inflow per 10m length</td><td>None</td><td><10 liters /min</td><td>10–25 lit./min</td><td>25 - 125 lit. /min</td><td colspan="3">>125 lit./min.</td></tr>
<tr><td>Joint water pressure/$\sigma 1$</td><td>0</td><td>0.1 – 0</td><td>0.1 – 0.2</td><td>0.2 – 0.5</td><td colspan="3">>0.5</td></tr>
<tr><td>General conditions</td><td>Completely dry</td><td>Damp</td><td>Wet</td><td>Dripping</td><td colspan="3">Flowing</td></tr>
<tr><td colspan="2">Rating</td><td>15</td><td>10</td><td>7</td><td>4</td><td colspan="3">0</td></tr>
<tr><td rowspan="4">6</td><td colspan="2">Joint orientation (dip/dip direction)</td><td>Very favorable</td><td>Favorable</td><td>Fair</td><td>Unfavorable</td><td colspan="3">Very unfavorable</td></tr>
<tr><td rowspan="3">Rating</td><td>Tunnel</td><td>0</td><td>-2</td><td>-5</td><td>-10</td><td colspan="3">-12</td></tr>
<tr><td>Foundations</td><td>0</td><td>-2</td><td>-7</td><td>-15</td><td colspan="3">-25</td></tr>
<tr><td>Slopes</td><td>0</td><td>-5</td><td>-25</td><td>-50</td><td colspan="3">-60</td></tr>
<tr><td colspan="3">Rock Mass Rating (RMR)</td><td>81-100</td><td>61-80</td><td>41-60</td><td>21-40</td><td colspan="3"><20</td></tr>
<tr><td colspan="3">Class no.</td><td>I</td><td>II</td><td>III</td><td>IV</td><td colspan="3">V</td></tr>
<tr><td colspan="3">Description</td><td>Very good rock mass</td><td>Good rock mass</td><td>Fair rock mass</td><td>Poor rock mass</td><td colspan="3">Very poor rock mass</td></tr>
<tr><td colspan="3">Stand up time (av.)</td><td>10 years for 15m span</td><td>6 months for 8m span</td><td>1 week for 5m span</td><td>10 hours for 2.5m span</td><td colspan="3">30 minutes for 1m span</td></tr>
<tr><td colspan="3">Cohesion of the rock mass (kPa)</td><td>>400</td><td>300 - 400</td><td>200 - 300</td><td>100- 200</td><td colspan="3"><100</td></tr>
<tr><td colspan="3">Friction angle of the rock mass</td><td>45°</td><td>35° - 45°</td><td>25° - 35°</td><td>15° - 25°</td><td colspan="3"><15°</td></tr>
</table>

Table No. 2.18: Adjustment Rating Factors for Joints and Slope Mass Rating, SMR (Romana, 1985)

Case	**Very favorable**	**Favorable**	**Fair**	**Unfavor-able**	**Very Unfavour - able**
Dip direction of (discontinuity – slope) $\lvert \alpha_j - \alpha_s \rvert$	>30°	30° – 20°	20° - 10°	10° - 05°	<05°
$\lvert \alpha_j - \alpha_s - 180° \rvert$ in case of toppling (T)	>30°	30° – 20°	20° - 10°	10° - 05°	<05°
F_1 for both plane failure (P) and toppling (T)	0.15	0.40	0.70	0.85	1.00
Joint dip $\lvert \beta_j \rvert$	<20°	20° - 30°	30° - 35°	35° - 45°	>45°
F_2 for plane failure (P)	0.15	0.40	0.70	0.85	1.0
F_2 for toppling (T)	1	1	1	1	1
Dip amount of (discontinuity–slope) $(\beta_j - \beta_s)$	>10°	10° - 0°	0°	0° - (-10°)	<-10°
Dip amount of (discontinuity+slope)$(\beta_j + \beta_s)$	<110°	110°-120°	>120°	--	--
F_3 for both plane failure (P) and toppling (T)	0	-6	-25	-50	-60
Methods of excavation	Natural slope	Presplitting	Smooth blasting	Blasting or Mechanical	Deficient blasting
Adjustment rating F_4	+15	+10	+8	0	-8
SMR value	0-20	21-40	41-60	61-80	81-100
SMR class number	**V**	**IV**	**III**	**II**	**I**
SMR Class Description	Very bad	Bad	Normal	Good	Very good
Stability Classes	Completely unstable (V)	Unstable (IV)	Partially stable (III)	Stable (II)	Completely stable (I)
Possible failures types	Big planer or soil like	Planer or big wedges	Some joints or many wedges	Some blocks	None
Support	Re-excavation	Important/corrective	Systematic	Occasional	None

2.2.2.3 Measurement of Discontinuities

A **line** in space is defined by its **trend** and **plunge**. Trend is the angle between the horizontal projection of the line and the north direction (Figure 2.16), whereas plunge is the angle between the line and its projection (Figure 2.16). Trend is measured on a horizontal plane and varies from 0 to 360 degrees, whereas plunge is measured on a vertical plane and varies from 0 to 90 degrees.

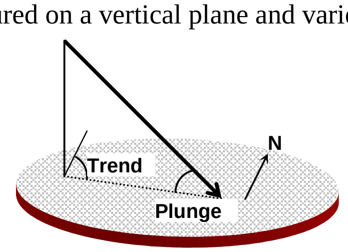


Figure 2.16: Definition of trend and plunge

A plane in space is defined by its **dip** and **strike**. The strike line is the horizontal line contained in a plane, whereas the dip line is the line of maximum slope that is perpendicular to the strike line and lies on the same plane (Figure 2.17). The direction of strike is the trend of the strike line, whereas the direction of dip is the trend of the dip line. On the other hand, the angle of dip is the plunge of the dip line. The plunge of the strike line is zero.

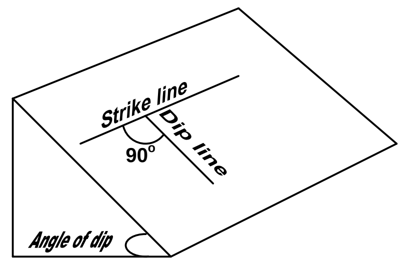


Figure 2.17: Definition of dip and strike

A geological compass permits one to measure the strike and *dip directions as well as the angle of dip* of an *inclined geological plane* and thus to define its position in space. In the case of a *vertical geological plane,* its *strike* defines this position. A *horizontal geological plane* has neither unique dip direction nor strike. Figure 2.18 further illustrates the above definitions.

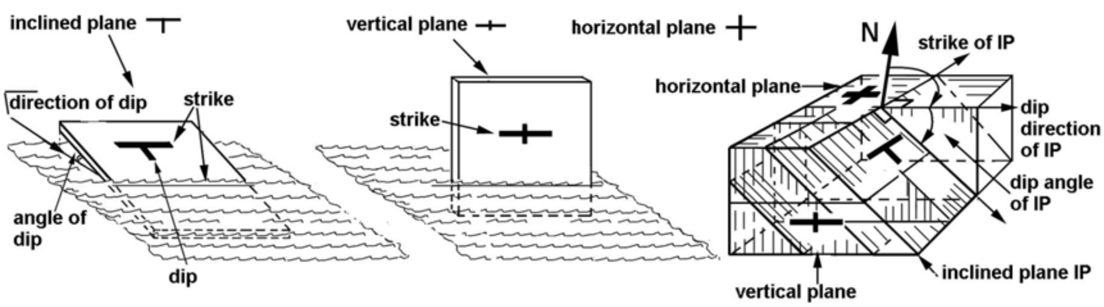


Figure 2.18: Strike Direction, Dip Direction, and Angle of Dip of a Geological Plane

The planes control the movement of water through the rock, and hence weathering. The angle **of dip (or simply dip)** is measured with a clinometer and **the bearing of dip (or dip direction)** is measured with a compass. The bearing can be any figure from 000° to 360°. It is generally written as a 3-figure number, e.g., 048. This distinguishes the bearing from the inclination, which cannot exceed 90°. A reading for the angle of dip greater than 90° means the slope is in fact dipping in the opposite direction.

As stated earlier**, strike** is the horizontal line contained in the plane of bedding, foliation, or jointing. The bearing of strike is measured with a compass. The figure is always given as a reading less than 180°. In practice, strike is measured first because a horizontal line needs to be established in order to find the maximum inclination of the dipping plane. Conventionally, the bearing of dip is written first, followed by the angle of dip, e.g., 115/35.

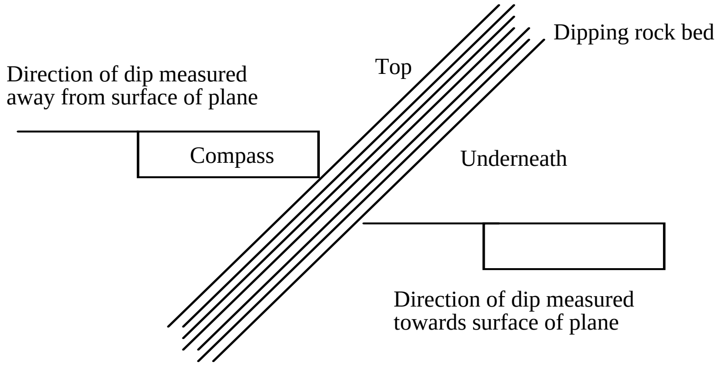


Figure 2.19: Measurement of Dip Direction and Angle of Dip of a Geological Plane

Note that the direction of dip is the direction towards which the plane is inclined. When you are measuring dipping surfaces in the field, the procedure for orienting the compass differs slightly according to whether you are measuring the plane from the top or underneath. If you are measuring from the top, the back of the compass is placed against the rock and you measure away from the plane. If you are measuring underneath, the front of the compass is placed against the rock and you measure into the plane.

2.2.2.4 The Graphical Analysis of Rock Structures

Introduction

Stereographic projection is one of the convenient methods of projecting the linear and planar features. This method is used exclusively for the determination of the angular relationship among the lines as well as planes. In geotechnical engineering, it provides a quick and reliable picture of the discontinuities and their intersections. It is also used for the estimation of cut slope angle, for the preparation of hazard maps, and for the estimation of safety factors.

Stereographic projection is a projection of the sphere. The sphere is divided into two equal hemispheres by a horizontal (equatorial) plane and the upper and lower poles are *fixed* as shown in Figure 2.20. The plane of projection is the equatorial plane itself. The circumference of the equatorial plane is called the primitive circle. For stereographic projection, one of the hemispheres is chosen. Here the technique of projection for the upper hemisphere is discussed. The principle of projection is the same for the lower hemisphere.

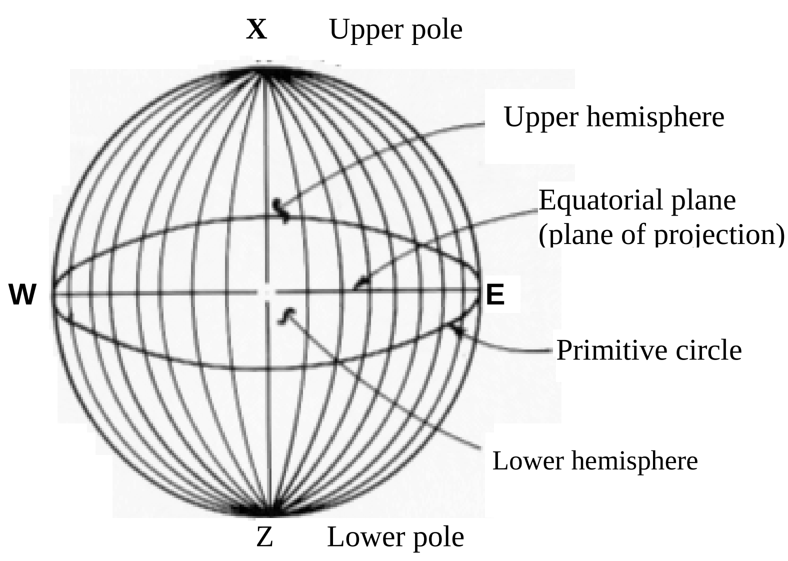


Figure 2.20: Projection Sphere

Projection of a Line

Any required line ℓ is moved parallel to itself in such a way that it passes through the centre of the sphere. In doing so, the line will pierce the sphere at two diametrically opposite points A and B (Figure 2.21). For the upper hemispherical projection, the upper point is projected down to the equatorial plane at the point of intersection *A'* of the line. The point A is joined with the lower pole Z (Figure 2.21).

If the line is vertical, the projection will be at the centre **0** of the sphere. If the line is horizontal, the projection will be at the primitive circle in diametrically opposite points *n'* and *n"* (Figure 2.22). All the inclined lines will be projected between the primitive circle and its centre (Figure 2.21 and 2.23).

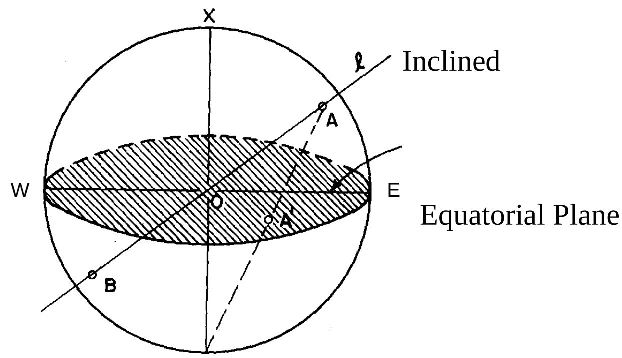


Figure 2.21: Projection of an inclined line on the upper hemisphere

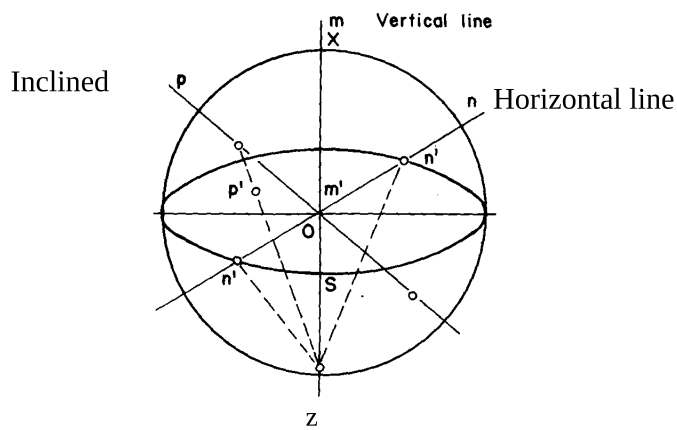


Figure 2.22: Projection of vertical, horizontal and inclined lines on the upper hemisphere

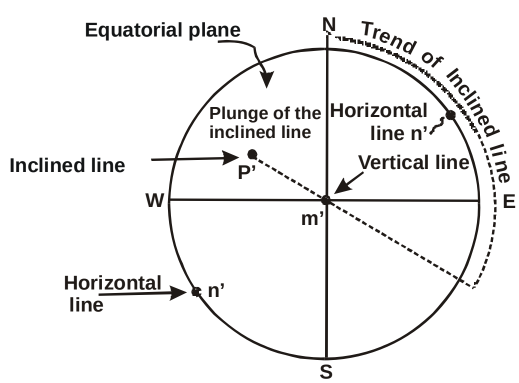


Figure 2.23: Stereographic projection of the horizontal, vertical and inclined lines of Figure 2.22

Projection of a Plane Line

A plane is projected in the same way as the line. We can imagine the plane as containing several lines that pass through the centre of the sphere. Each of the lines is projected on the equatorial plane and their trajectory will give the projection of the plane.

In general, a plane P is moved parallel to itself unless it passes through the centre of the sphere. The intersection of the plane with the sphere is a great circle with the radius equal to that of the sphere or primitive circle (Figure 2.24). Then each of the points k, 1, m, n, etc. from the upper hemispherical part of the great circle (Figure 3.24*a)*, is joined by straight lines with the lower pole Z and the trace of the intersecting points *k", ii', m', n',* etc. on the equatorial plane gives the projection (Figure 2.25 and 2.26).

Any vertical plane passing through the centre will bisect both the hemispheres. Its projection will be a straight line passing through the centre. Any horizontal plane passing through the centre is projected as the primitive circle itself. Inclined planes are projected as curves known as great circles with the ends at diametrically opposite points in the primitive circle (Figure 2.26).

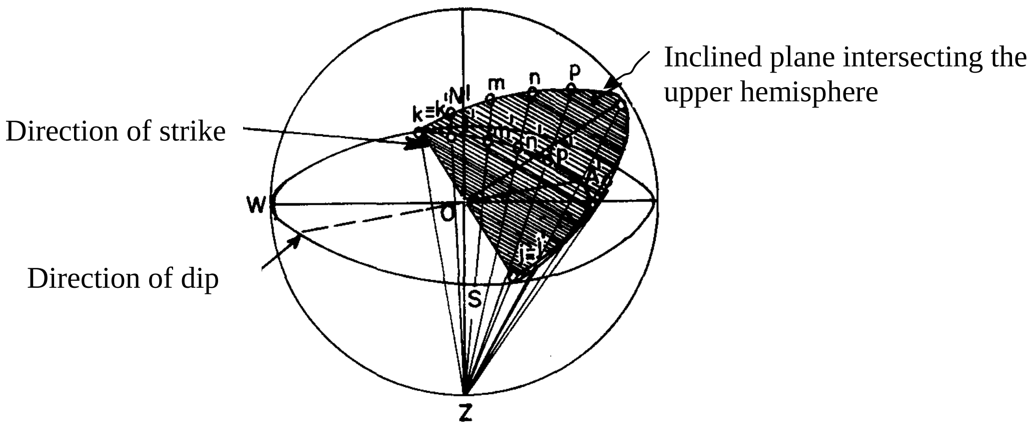


Figure 2.25: Projection of an inclined plane, intersecting the upper hemisphere

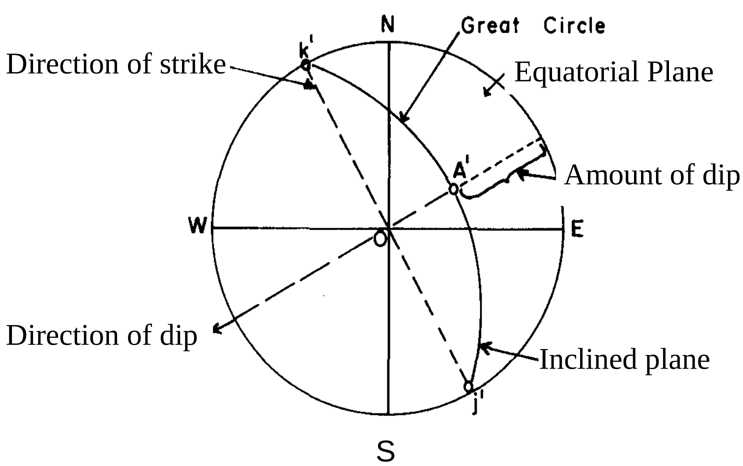


Figure 2.26: Stereographic projection of an inclined plane in Figure 2-34

Projection of a Cone

Let us consider a cone with a vertex at the centre of the sphere and horizontal axis of rotation (Figure 2.27).

It will intersect the sphere in a circle (Figure 2.27). By joining all the points of the circle from the upper hemisphere, we get a projection of the cone as small circles (Figure 2.28).

By constructing a series of great circles and small circles on the upper hemisphere, we obtain the Meridional Stereographic, or Wulif Net, or Stereonet. In it the two types of curves are generally drawn at 2° intervals (Figure 2.29).

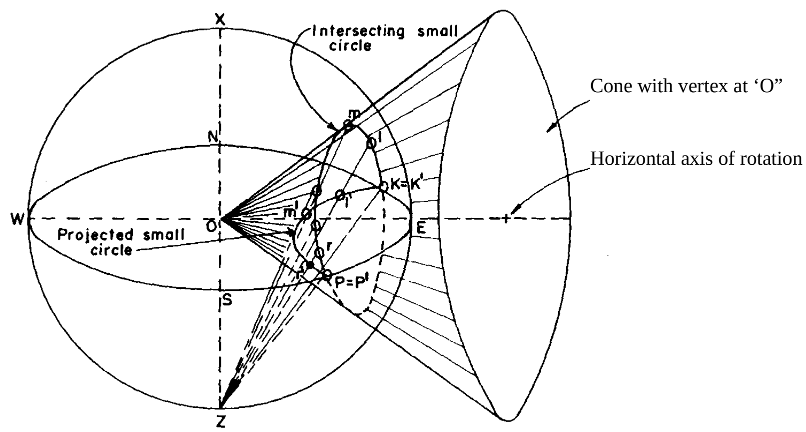


Figure 2.27: Projection of a cone on the upper hemisphere

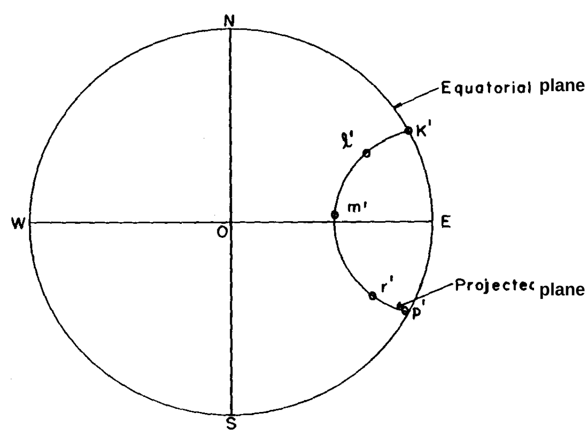


Figure 2.28: The stereographic projection of the cone

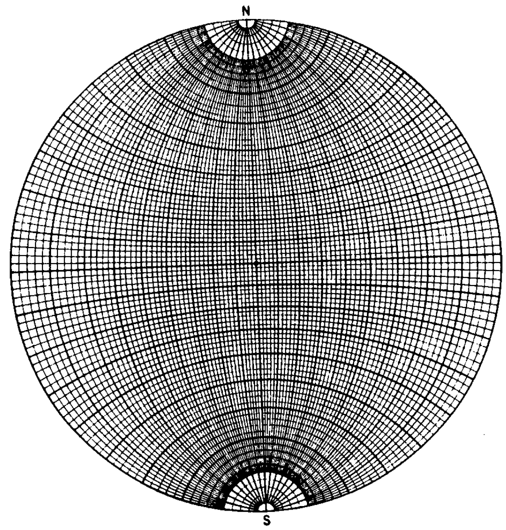


Figure 2.29: Wulff Stereographic Net

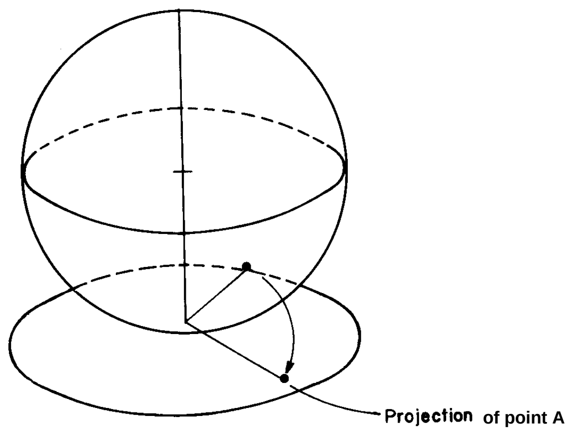


Figure 2.30: Method of projection for equal-area net

Figure 2.29 shows that the area on the net, for example, 10°x l0° in the centre of the net, is smaller than the same area at the margin. It creates problems when doing a statistical analysis of the data, as the randomly distributed points on the Wulff Net will show a concentration of points at the centre. That is, the random line would falsely show a weak preferred orientation in the vertical position. To overcome this problem, a somewhat different type of projection is needed. This method of projection is called Lambert Equal Area, or simply Equal Area Projection. (Figure 2.30). The small and great circles of the Wulff Net are modified to a series of curves and the result is the equal area or Schmidt Net (Figure 2.31). Our further investigations will be carried out exclusively on the Schmidt Net in order to avoid confusion.

Plotting Techniques of Linear And Planar Features of a Rock

When plotting on the stereo net, it is important to visualise the net as the convex-upwards hemisphere

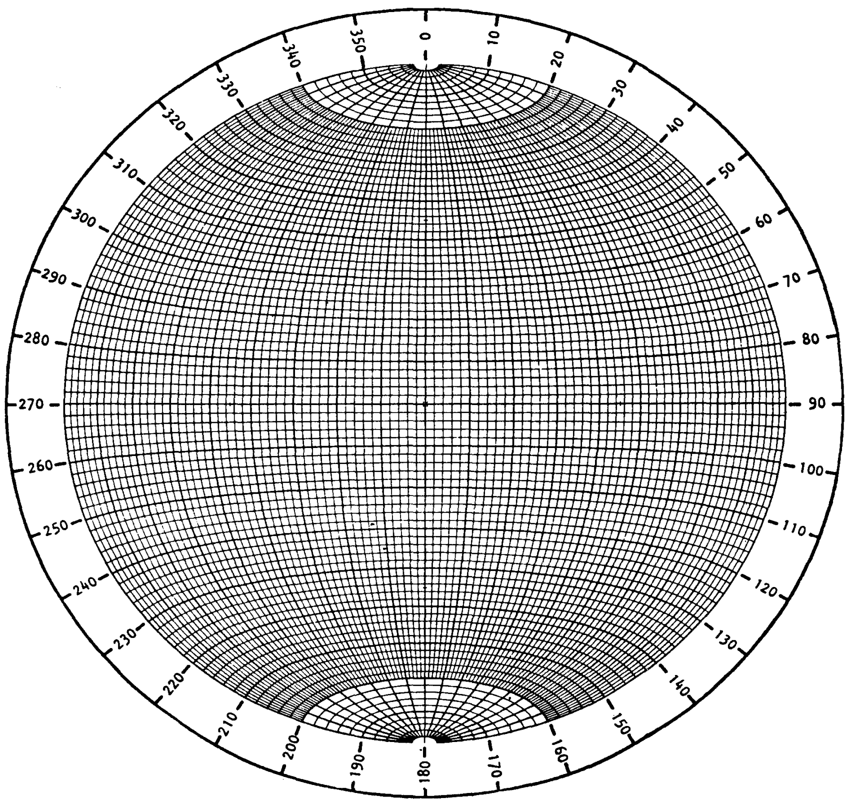


Figure 2.31: Equal Area or Schmidt Net

and to imagine the curves inscribed on its outer surface. To make the plotting visual, several Figures are given together with the plotting on the Equal Area Net.

Plotting A Line (Figure. 2.32)

Given a line with trend = 40° and plunge = 46°**:**

1) with the tracing paper on the Equal Area Stereo net, trace the primitive circle and make a small tick mark and label it N; this is the first step in all works on the stereo net;
2) to locate the trend of the line, count off 40° clockwise from N and make another small mark on the primitive circle at this point;
3) revolve this trend mark about the centre of the net to the north point (or any other straight line such as East-West and North-South) of the net;
4) count off 46° from the primitive, inwards along the diametrically opposite (Southern) end of the North-South straight line and mark the point P; and

5) restore the overlay to the starting position and recheck by visualization.

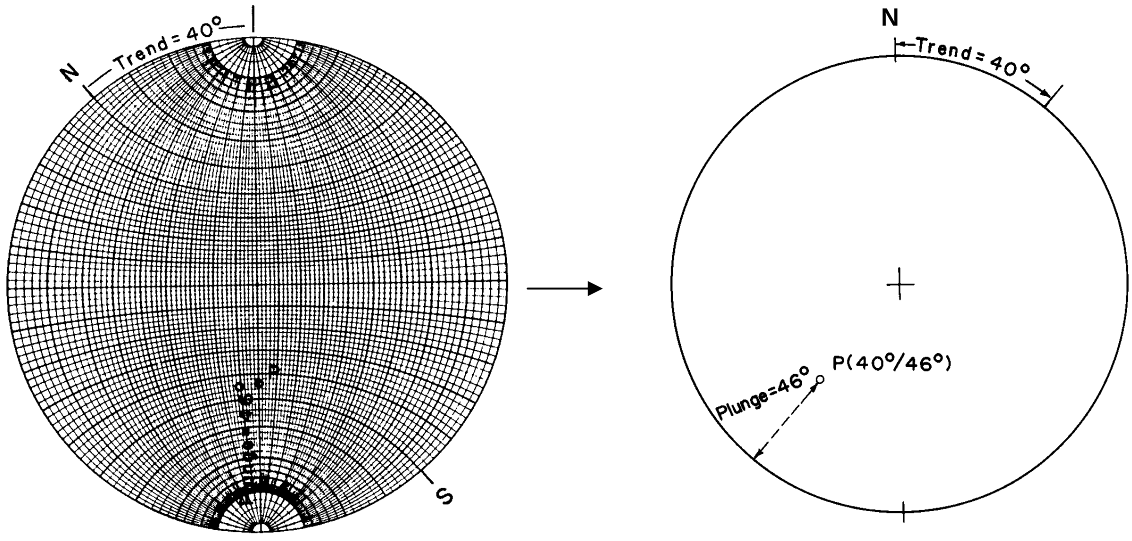


Figure 2.32: Plotting a Line

6) Visualization: Hold a pencil over the centre of the stereo net in the direction of the given trend direction at the angle of plunge. Visualize its intersection with the upper hemisphere in the southwest quadrant.

Plotting A Plane (Figure 2.33)

Given a plane with dip direction = 324° and angle of dip = 40°

1) With the overlay in place (Figure 2.33a) and the north index marked N, locate a point on the primitive representing the dip direction of the plane by counting 324° clockwise or 36° counter clockwise (Figure 2.33b).

2) Turn the dip direction mark on the overlay about the centre of the net (Figure 2.33c) until it is

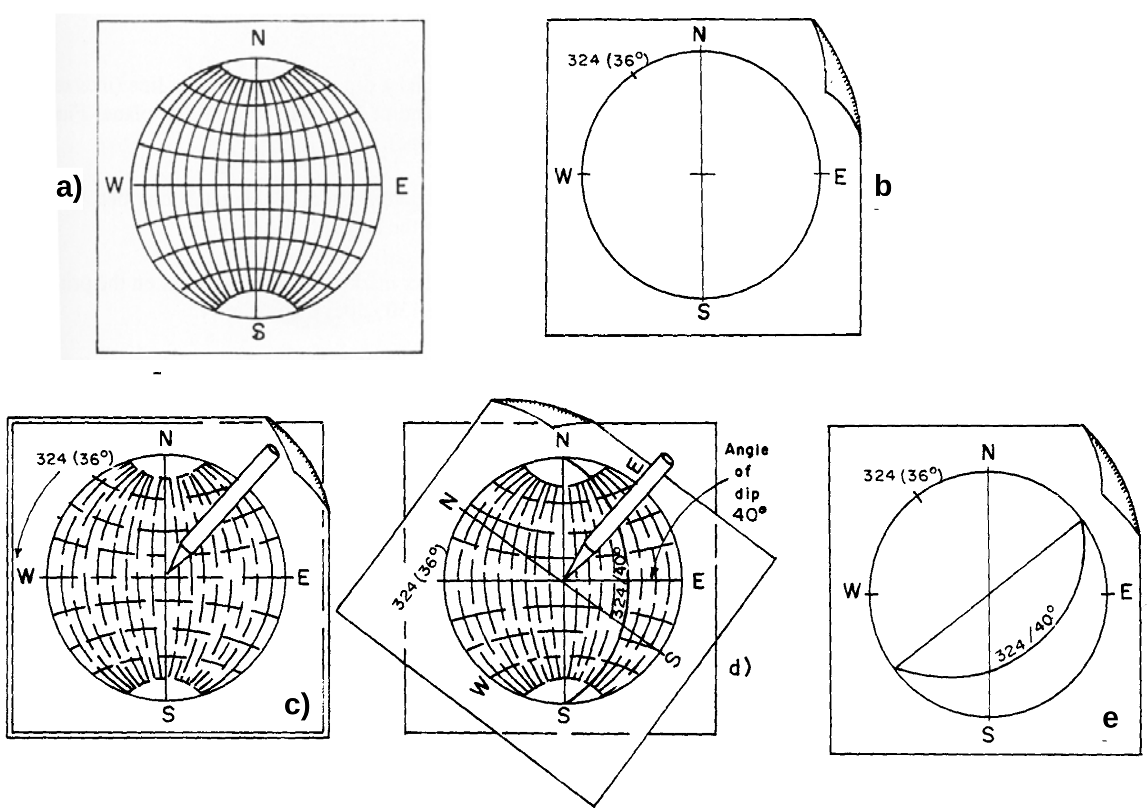


Figure 2.33: Plotting a Plane

exactly over the west (or east) direction (Figure 2.33d). Count off 40° from the primitive, inwards along the diametrically opposite end (i.e., the eastern end) of the East-West straight line of the net, and trace the required great circle (Figure 2.33 d).

3) Restore the overlay to the starting position and recheck by visualization (Figure 3.33 e).

Remarks: Do not forget to begin counting off inwards from the diametrically opposite end of the marked position. If you count off directly inwards from the marked position, it will be the projection on the lower hemisphere. Visualization: Hold the right hand palm upwards over the centre of the stereo net with the fingers pointing towards 324° + 90° = 414° = *54°* NE and the plane of the hand inclined 40° to the northwest (324°). The plane of the hand in this position can be imagined to extend into the upper hemisphere and intersect its surface. Its trace cuts the southeast quadrant and this is where the final plot must be.

Plotting a Line Contained in a Plane

Given a plane (sandstone bed) with a dip direction of 130°, a dip amount of *55°*, and a line (intersection of the vertical cut slope with the sandstone bed) with a trend of 78° and lying in that plane, find the plunge of the line (apparent dip of the bed along that cut slope).

Note: The apparent dip is the angle between the line of intersection of the sandstone bed with the vertical cut slope and any horizontal line parallel to the slope.

1) With the overlay in place (Figure 2.34), and the north index marked N, locate a point on the primitive representing the dip direction of the plane by counting 130° clockwise from N.
2) Turn the dip direction mark about the centre of the net until it is exactly over the east (or west) direction (Figure 2.34). Count off *55°* from the primitive, inwards along the diametrically opposite end (i.e., the western end) of the East-West straight line, and trace the required great circle.
3) Restore the overlay to its starting position and count off 78° clockwise from N and make a tick mark.
4) Rotate the overlay about the centre and bring the tick mark together with the nearby diameter and locate the required point (line) in the previously traced great circle. In the same position, count off the plunge of the line which is equal to the angle between the tick-marked point in the primitive and point (line) located in the great circle.
5) Restore the overlay to the starting position and recheck by visualisation (Figure 2.34).

Result: The plunge is 41°.

Plotting a Pole

Like any other projection, stereographic projection reduces the dimensions of the object by one. Here, a line becomes a point, and the plane is reduced to a curve. It is important to note the fact that every plane has a unique normal which can define its angular relations unequivocally. Therefore, it becomes possible to reduce further the dimension of the plane and to represent it on the stereo net by a point called the pole. To visualize it, hold the left hand as in Figure 2.33 c, d and hold a pencil between the fingers so that it is perpendicular to the plane of the hand. The line of the pencil will intersect the upper hemisphere at a point in the northwest quadrant.

Given a plane with a dip direction of 60° and a dip angle of 72°, plot the plane and locate the pole (Figure 2.35).

1) With the overlay in place and the north index marked N, locate a point on the primitive circle equal to the dip direction of the plane by counting off 60° clockwise from N and tick mark it.
2) Rotate the overlay about the centre until the tick mark is over the East-West diameter of the net, and by counting off 72° inwards from the opposite end of the diameter, trace the required great circle.

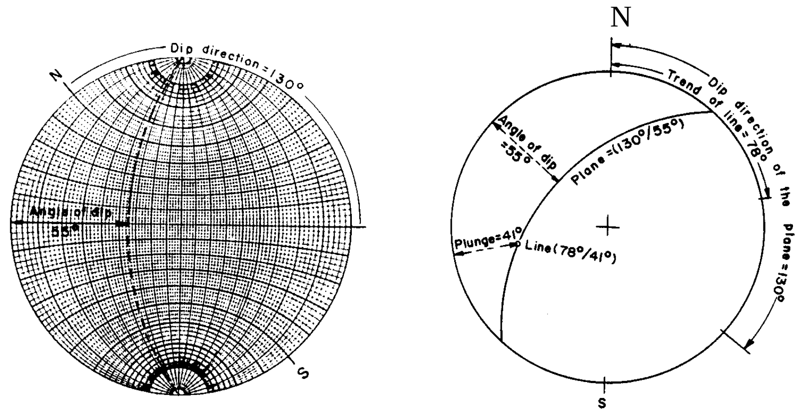


Figure 2.34: Plotting a line contained in a plane

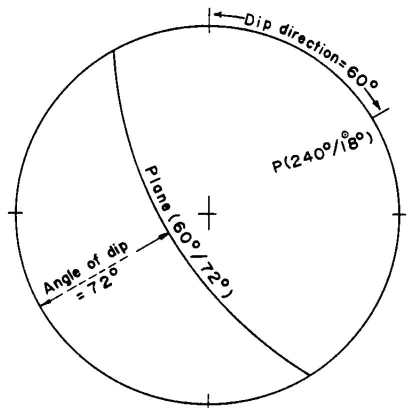


Figure 2.35: Plotting the pole to a plane

3) As the pole is everywhere located 90° from the plane, count off 90° towards the tick-marked point from the great circle and locate the pole (Figure 2.34).

3) Restore the overlay to its original position and visualize the plotting by holding the left hand over the centre of the net and tilting it towards the NE (60°) by 72°, while the fingers will point towards 60°-90° = -30° = 330° NW (the strike). In this position, the plane will be plotted in the SW quadrant and the pole in the NE quadrant.

Note: It is easier to plot the pole by rotating the overlay so that the dip direction mark coincides with any nearby diameter and by counting off the dip amount (here, 70°) from the centre of the net towards the tick mark (dip direction). Check the previous results with this method.

Result: The position of the pole is 240°/18°.

Plotting the Line of Intersection of Two Planes (Figure 3.36)

Given two intersecting planes (joints), having dip amounts of 40° and 60° and a dip direction of 120° and 260° respectively, find the plunge and the trend of the line of intersection (wedge axis).

1) With a tracing paper over the stereo net, trace the primitive circle and mark the north point. Measure off the dip direction of 120° clockwise from N and mark this position on the primitive.

2) Rotate the overlay about the centre of the net until the dip direction mark lies on the East-West

diameter of the net. Measure 40° from the primitive and trace the required great circle.

3) Rotate back the tracing paper to its original position and repeat the same for the second plane.
4) The point of intersection of the two great circles defines the required line. Rotate the overlay until the intersection of the two great circles lies along the East-West diameter of the stereo net and measure the plunge of the line of intersection. Also, mark with a tick on the overlay at the diametrically opposite direction (trend) along the East-West line on the primitive circle.
5) Rotate the tracing back to its original position and count off the trend of the line.

Result: The trend and plunge of the line are 183° and 22° respectively.

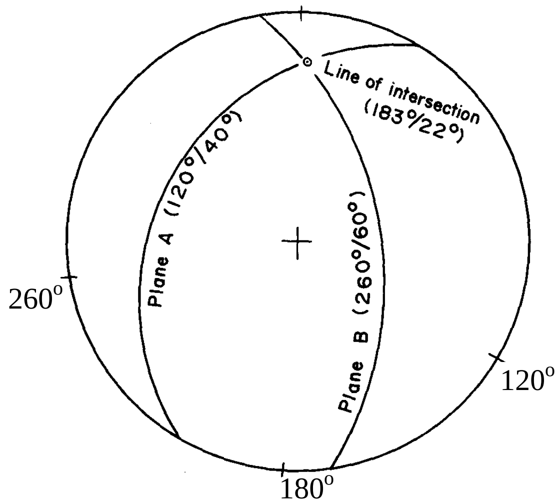


Figure 2.36: Plotting the line of intersection of two planes

2.2.2.5 The Hill Slope Processes and Slope Stability Analysis

a. Hill Slope Processes in Nepal

Hill slopes with steep gradients are the major physiographic features of Nepal. High gradient slopes providing high energy for the active hill slope process and our design as well as the selected site should be sustainable in spite of the process.

The pathways and effects of water movement over and through the ground, and the movement of water over the surface, into the surface to a depth of a few centimetres, further down into the soil profile, and deep into rocks can lead to instability in various forms. Each situation will be dealt with in turn and their relevance to mass movement.

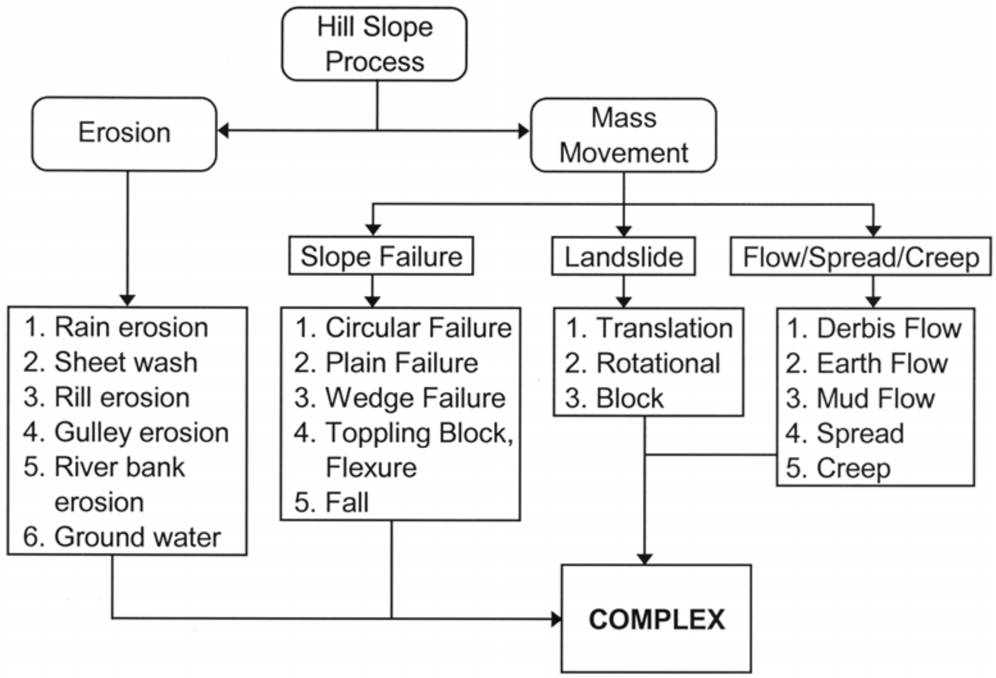


Figure 2.37: Flow Chart of the Common Hill Slope Processes

b. Role of Surface and Ground Water

Ground Conditions Leading to Overland Flow of Water

Conditions that lead to overland flow are:

- when the soil has a capping (compacted surface). A soil cap will prevent infiltration even if the soil itself is highly permeable;
- when the rate of precipitation exceeds that of infiltration (when the soil is not saturated);
- when the soil is saturated;
- when impermeable rock or impermeable soil is at the surface;
- slope, to a limited extent, can determine whether or not overland flow takes place. If the slope is very steep, water will flow over it however permeable the surface is. However, for most practical situations, slope does not *cause* overland flow, although it certainly influences the *rate* of overland flow.

Erosion Resulting From Surface Water Movement

Sheet erosion is the removal of soil, mineral or rock particles evenly over the whole surface. Sheet erosion persists on firm ground that resists riling, e.g., very weak rocks. Mudstone, soft sandstone and Siwalik rocks are typical examples of this material. A thin weathered skin develops on these rocks, that is then removed by rain-wash to expose firmer rock beneath. Thus, softening and removal of the rock continues only on the surface. Rill erosion is the removal of soil, mineral or rock particles along water channels. This is by far the commonest form of erosion. As rills become larger and deeper, they develop into gullies. Fine soil of low cohesion, e.g., silt or silty soil, is most susceptible to rill erosion. This is because fine, non-cohesive particles are very easily detached by water and carried in suspension.

Sub-Surface Water Movement and Related Mass Movement

When water percolates into the soil, it enters the voids and starts to fill them up. As a result, pore water pressure starts to rise. Pore water pressure is the pressure acting on soil grains by water held in the pores. Pore water pressure can be positive or negative. It is negative when the voids are only partially filled with water. (This state is also known as soil suction). Pore water pressure becomes neutral just before the point at which the voids become completely filled with water. Pore water pressure becomes positive at the point when all the air has been expelled from the voids and the water phase in the soil-water mix becomes continuous. At that point, the water phase becomes a column and hydrostatic pressure, equivalent to the height of the column, is exerted within the pores. The pressure is transferred to the soil grains.

If the hydrostatic pressure is sufficiently high, it will force the grains apart and the mixture will start to behave as a liquid. Hydrostatic pressure which develops near the soil surface, as when the upper layer becomes saturated during a heavy rain, causes the soil to flow.

When pore water pressure becomes positive along the walls of a fissure underground, a "pipe" develops. A pipe is an enlarged fissure that forms underground in fine-grained, non-cohesive soil, especially silty or fine sandy soils. Enlargement of the fissure takes place when water, which is flowing along the fissure or into the fissure from the side walls, detaches particles of soil and carries them away in suspension.

Pipes that have not broken through to the surface can still sometimes be detected by the presence of an elongated hollow of subsided ground pointing down the slope. The trench may be above the head of a gully and in the same alignment as the gully, indicating that water is moving into the gully head as ground water through a pipe.

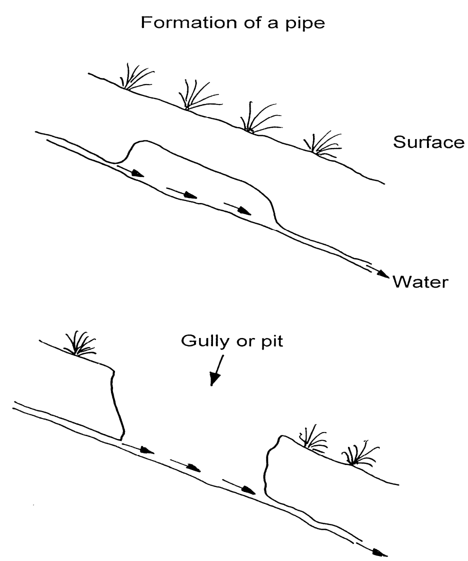


Figure 2.38: Sub-Surface Water Movement and Related Mass Movement

If water travels downward to the bottom of the soil profile, it is commonly stopped in its path by the impermeable surface of the rock beneath. It then migrate
s downhill along the interface until it emerges as a spring at a point where the soil becomes shallower or the rock outcrops at the surface. Pore water pressure may become positive at the base of the soil profile, resulting in a deep transnational landslide (the commonest deep type) or circular failure.

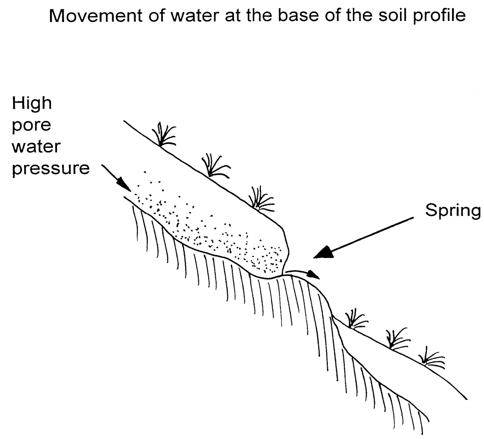


Figure 2.39: Sub-Surface Water Movement and Related Mass Movement

Water in Rocks

If water goes deeper than the soil profile, it goes into the bedrock. In horizontal rocks, it will slowly move sideways along the surface of an impermeable layer. In tilted beds, it will move more rapidly down the slope. If the rock is fractured, the water will continue to go deeper. Hydrostatic pressure is exerted within the open joint systems of rocks in exactly the same way as in soils. If the water cannot escape as spring water, high pressure can develop and force the joints apart. This is the cause of many rockslides.

Water movement through rocks is controlled by:

- the permeability of the rock;
- the angle of bedding;
- the number, orientation, openness and continuity of fractures.

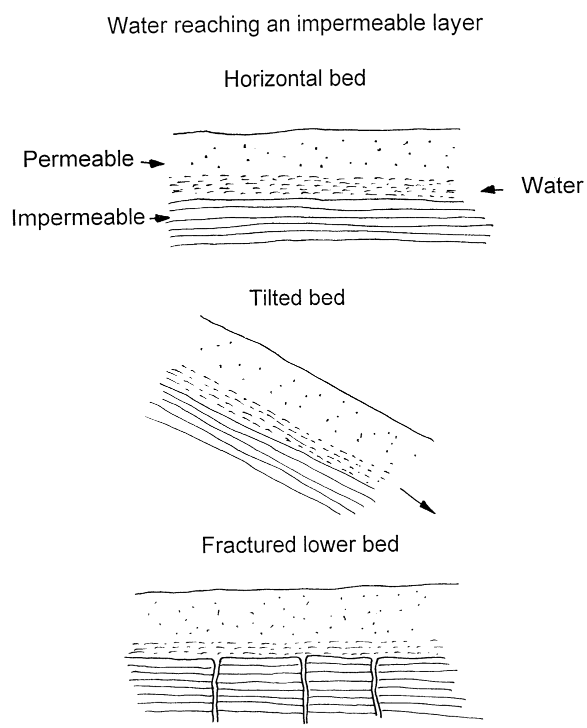


Figure 2.40: Sub-surface water movement and related mass movement

Conclusion

There is a direct relationship between site information like external signs connected with water on a slope and the movement of water through the slope with the causes of instability on a slope. To gather this information while assessing slope instability, it is necessary to:

- identify types of materials on a site that allow easy water movement or restrict this movement;
- identify ground surface features which indicate points where water moves into the slope and the points where it emerges;
- identify ground surface features which indicate specific types of instability.

Nepalese soils and rocks are generally very permeable, containing voids and many fractures. These allow water into various depths, all of which can cause instability of various kinds.

In order to identify the specific mitigation measures, an analysis of these forms of instabilities in terms of failure mechanism is essential. These will be discussed in subsequent sessions.

c. Failure Mechanism

There are many classification schemes for mass movements proposed by different authors like Campbell (1951), Hutchison (1968, 1969, 1977), Crozier (1973) and Varnes (1958, 1978). Hutchinson's classification considers movement criteria including depth, direction and sequence of movement with respect to the initial failure. Varnes (1978) Classification is based on nature of source material and the type of movement.

Types of Landslides according to Varnes

The types of landslides proposed by Varnes (1978) is the most commonly used in the world. It is also adopted by the Landslide Committee, Highway Research Board, Washington, D.C. It divides landslides into falls, topples, slides, lateral spreads and flows. Wherever two or more types of movements are involved, the slides are termed as complex. Varnes (1978) has divided the materials prone to landslides into classes, e.g. rock and soil. The soil is again divided into debris and earth (Table 2.19).

Table No. 2.19: Types of Landslides (Varnes, 1978)

Type of movement			**Type of Material**		
			Engineering soils		**Bedrock**
			Predominantly fine	**Predominantly coarse**	
Falls			Earth fall	Debris fall	Rockfall
Topples			Earth topple	Debris topple	Rock topple
Slides	Rotational	Few Units	Earth slump	Debris slump	Rock slump
	Translational	Few units Many units	Earth block slide Earth slide	Debris block slide Debris slide	Rock block slide Rock slide
Lateral spreads			Earth spread	Debris spread	Rock spread
Flows			Earth flow	Debris flow	Rock flow
			(Soil creep)		(Deep creep)
Complex			Combination of two or more principal types of movement		

Falls

Falls are abrupt movements of the slope material that becomes detached from steep slopes or cliffs. Movement occurs as free-fall, bouncing or rolling. Depending on the type of materials involved, the result is a rock fall, soil fall, debris fall, earth fall, boulder fall, and so on. The typical slope angle of occurrence of falls is from 45-90 degrees. Undercutting, differential weathering, excavation, or stream erosion promotes all types of falls.

Topple

A topple is a block or a series of blocks that tilts or rotates forward on a pivot or hinge point and then separates from the main mass, falling to the slope below, and subsequently bouncing or rolling down the slope.

Slides

Although many types of slope movements are included in the general term “landslide”, the more restrictive use of the term refers to movements of soil or rock along a distinct surface of rupture which separates the slide material from more stable underlying material. The two major types of landslides are rotational slides and transnational slides.

Rotational Slides

These slides refer to a failure, which involves sliding movement on a circular or near circular surface of failure. They generally occur on slopes of homogeneous clay, deep weathered and fractured rocks and soil. The movement is more or less rotational about an axis that is parallel to the contour of the slope. A scarp at the head characterizes such slides, which may be nearly vertical. These slides may be single rotational, multiple rotational or successive rotational types. Accordingly, they may have a single surface of rupture or multiple surface of rupture. A “slump” is an example of a small rotational slide.

Translational Slides

These are non-rotational block slides involving mass movements on more or less planar surfaces. Translational slides are controlled by weak surface such as beddings, joints, foliations, faults and shear zones. The slide material involved may range from unconsolidated soils to extensive slabs of rock and debris. Block slides are transitional slides in which the sliding mass consists of a single unit or a few closely related units of rock block that moves down a slope. Translational slides may progress over great distances.

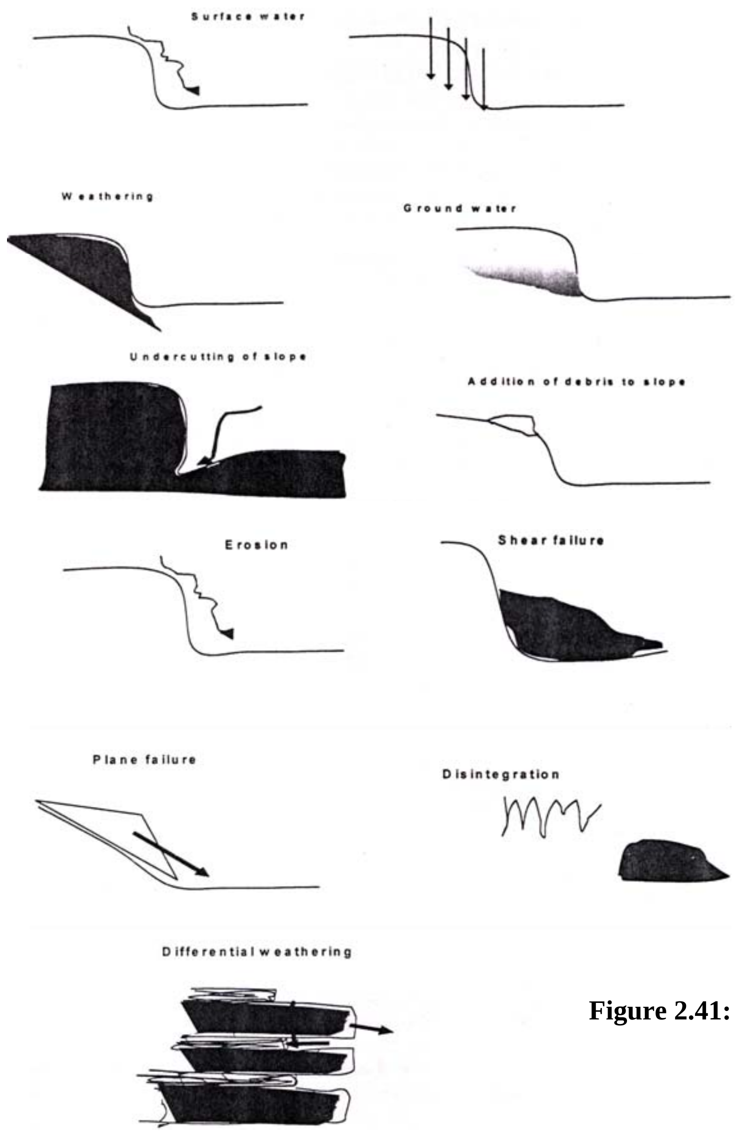


Figure 2.41: Different types of mass wasting phenomena

Lateral Spreads

Lateral spreads are a result of the nearly horizontal movement of unconsolidated materials and are distinctive because they usually occur on very gentle slopes. The failure is caused by liquefaction, the process whereby saturated, loose or non-cohesive sediments (usually sands and silts) are transformed from a solid into a liquefied state, or plastic flow of subjacent material. The failure is usually triggered by rapid ground motion such as that experienced during an earthquake, or by slow chemical changes in the pore water and mineral constituents.

Flows

There are several types of flows, and a short description of them is given below.

Creep

Creep is the imperceptibly slow, steady downward movement of slope-forming soil or rock. Curved tree trunks, bent fences or retaining walls, tilted poles or fences, and small ripples or terracettes indicate creep.

Debris Flow

Debris flow is a form of rapid mass movement in which loose soils, rocks and organic matter combine with entrained air and water to form slurry that then flows down-slope. Debris flow areas are usually associated with steep ravines where there are some active landslides. Individual debris flow areas can usually be identified by the presence of debris fans at the termini of the drainage basins. In general, the following conditions are important for the formation of debris flow:

- Slopes with 20-45 degrees
- Saturated loose rock and soil materials with high content of clay minerals
- High intensity and duration of rainfall

Debris Avalanche

Debris avalanche is a variety of very rapid to extremely rapid slide-debris flow process.

Earth Flow

Earth flow has a characteristic “hourglass” shape. A bowl or depression forms at the head where the unstable material collects and flows out. The central area is narrow and usually becomes wider as it reaches the valley floor. Earth flows generally occur in fine-grained materials or clay-bearing rock on moderate slopes with saturated conditions. However, dry flows of granular material are also possible.

Mudflow

Mudflow is an earth flow that consists of material that is wet enough to flow rapidly and that contains at least 50 per cent sand-, silt- and clay-sized particles.

Complex Movements

Complex movement is a combination of two or more types of movements mentioned above. Generally, huge-scale movements are complex, such as rock fall, rock/debris avalanches. The characteristic features of the different types of landslides are simply illustrated in Figure 2.43.

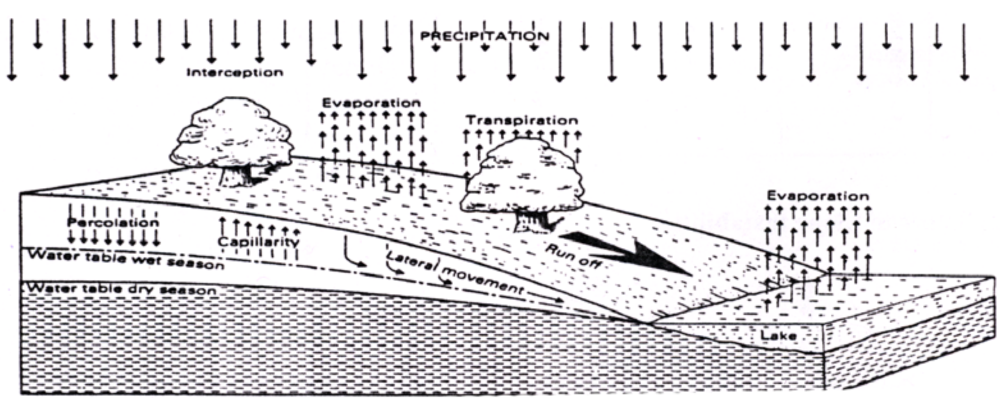


Figure 2.42: Water Movement and Related Mass Movement Phenomena

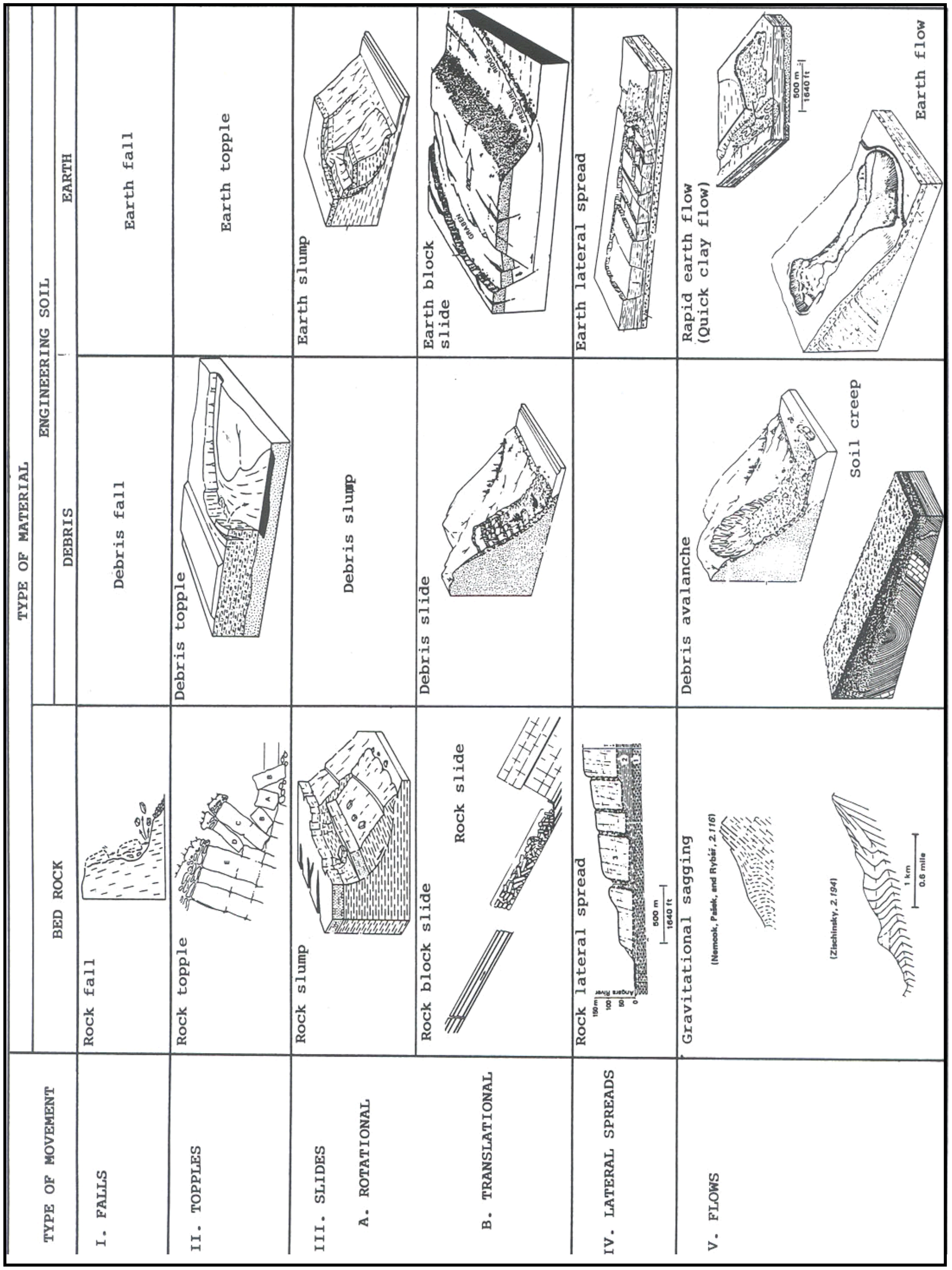


Figure 2.43: Types of Slope Instabilities

d. Slope Stabilization and Protection Measures

Slope Protection
Rock slope

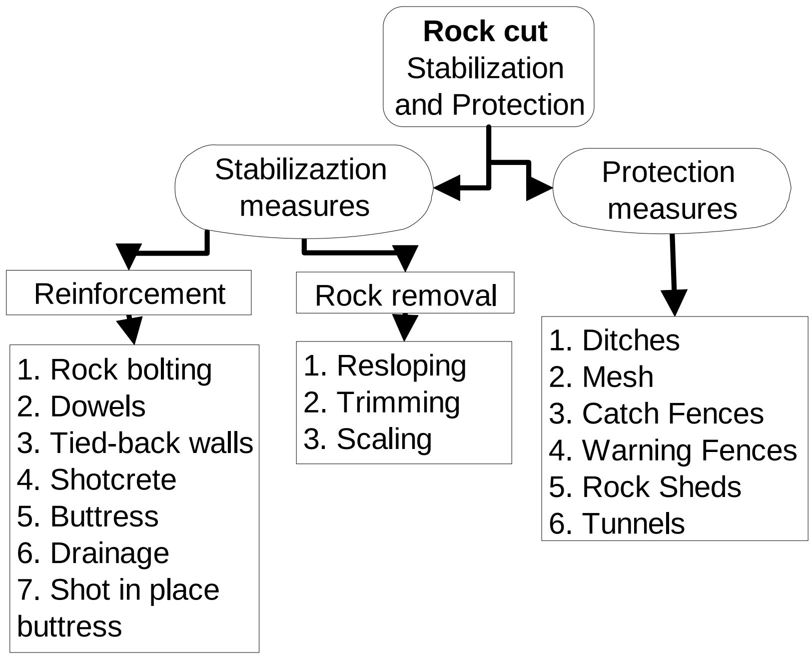


Figure 2.44: Rock cut slope stabilization and protection measures

i. Reinforcement

The main objective of rock reinforcement techniques is to minimize the relaxation and loosening of a rock mass that may take place as a result of excavation and unloading. Disaster managers are familiar with the fact that once the relaxation of a rock mass has taken place, it is not possible to reverse the process. Reinforcement is essential to maintain the interlocked rock mass and to prevent significant decrease in shearing strength.

ii. Rock Removal

Stability of rock slopes can be maintained by removing potentially unstable rock. Rock removal is a preferred method of stabilization as it eliminates the hazard and no future maintenance is required, but in certain circumstances, rock removal should not be done. Typical removal methods including the following are mentioned in Figure 2.45.

Resolving zones of unstable rock

Trim blasting overhangs and

Scaling individual blocks of rock.

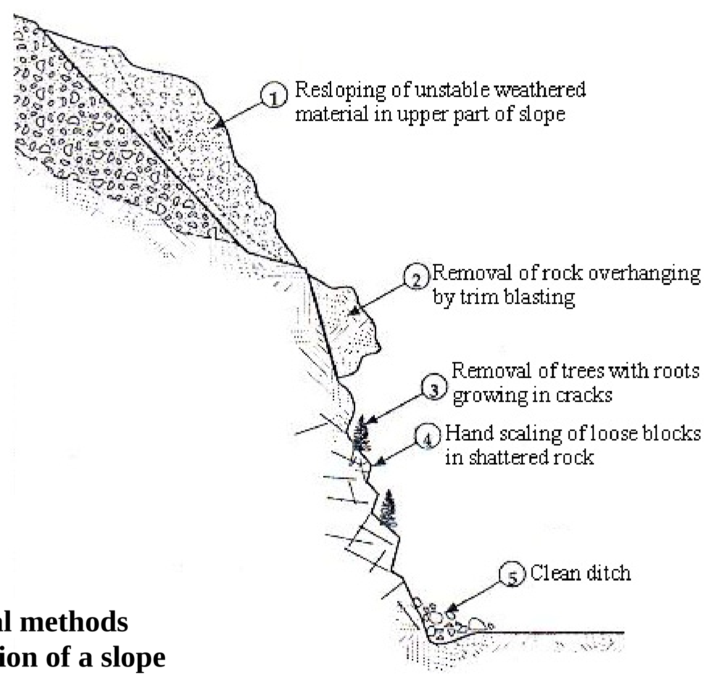


Figure 2.45: Rock removal methods for stabilization of a slope

Protection Measures

Protection of rock slopes against instability includes catchment ditches and barriers, wire mesh fences, mesh hung on the face of the slope and rock sheds. Common features of the protection measure are their energy absorbing characteristics, which either stop a rock fall over some distance or deflect it away from facilities.

Benched Slopes

Excavation of intermediate benches on rock cuts are sometimes useful as a protection measure. When the narrow benches fill up with debris, they will no more be effective in catching further rock falls. Maintenance of such narrow benches is rarely possible.

Ditches

If space is available, a catch ditch at the toe of the slope is often a cost-effective means of stopping rockfalls. The required dimensions of the ditch as defined by the width of the base and the depth are related to the high and face angle of the slope. In case of steep slopes (> 75^0) the rocks tend to stay close to the face and land near the toe of the slope, whereas in case of slope with a steepness of 55^0-75^0, the rocks tend to bounce or spin so that they travel considerable distances from the toe thus requiring a wide ditch. In slopes of 40^0 to 55^0 steepness, the rock blocks tend to roll down the face and into the ditch and a steep outer face is required to prevent them from rolling out. Slopes with protrusions on the face demand increased dimensions of the ditches.

Barriers

A variety of barriers can be constructed as protection measures. Selection of the appropriate type of barrier depends on the energy of the falling blocks, the slope dimensions and the availability of construction materials. Gabions, concrete blocks or geo fabric and soil barriers are commonly known to disaster managers.

Rock Catch Fences Attenuators

The design of such fences and attenuators for a particular site depends on the topography, anticipated impact load, bounce height and availability of material. Catch fences and attenuators also exhibit energy absorbing characteristics along with the absence of any rigid components. The falling blocks collide with the net of the fences and sometimes deformation of the mesh is also possible. The energy of the falling block may be absorbed during collection. Woven wire rope nets and Flex-post rockfall fences are the most common rock catch fences prescribed by disaster managers.

Soil Slope

Table No. 2.20: Approaches to stability problems (modified from Gedney and Weber 1978)

Category	Procedure	Best Application	Limitation	Remarks
Avoid problem	Relocate bridge	As an alternate anywhere	Large cost if redesign	Requires detailed studies
	Completely or partially remove unstable materials	Where small volumes of excavation are involved and where poor soils are encountered at shallow depths	May be costly	Requires detailed studies Depth of excavation must be sufficient
Reduce driving forces	Drain surface	In any design scheme, must also be part of remedial design	Correct only surface infiltration or seepage due to infiltration	Slope vegetation should be considered
	Drain subsurface	On any slope where lowering of ground water table will increase slope stability	Cannot be used effectively when sliding mass is impervious	Stability analysis should include consideration of seepage
	Reduce weight	At any existing or potential slide	Requires light weight materials, which may be costly	Stability analysis must be performed to ensure proper placement of light weight materials

Category	Procedure	Best Application	Limitation	Remarks
Increase resisting forces	Use buttress and counterweight fills; toe berms	At any existing landslide: in combination with other methods	May not be effective in deep seated landslides Must be founded on a firm foundation	Consider reinforced steep slopes
Apply external force	Use structural analysis	To prevent movement before excavation	Will not stand large deformations; must penetrate well below sliding surface	Stability and soil structure analysis is required
Increase internal force	Drain subsurface	At any landslide where water table is above shear surface	Requires experienced person to install and ensure effective operation	Detailed study is required
Strength	Use reinforced backfill	On embankments and steep fill slopes	Requires long term durability of reinforcement	
	Install in situ reinforcement	As temporary structures in stiff soil		
	Use biotechnical stabilization	On soil slope of modest height	Climate; may require irrigation in dry season; longevity of selected plants	Design is by trial and error plus local experience
	Treat chemically	Where sliding surface is well defined and soil reacts positively to treatment	May be reversible; environment stability is unknown	Laboratory study is required

e. Riverbank Protection

Alluvial Riverbank Erosion

In curved channels, the top flow lines, which contain less sediment concentration but have more velocity, move towards a concave bank, whereas the bottom flow lines with sediment laden water but with less velocity separate themselves from the top flow lines and move towards a convex bank. The net effect remains such that a helical flow takes place (anti-clockwise if the concave bank is on the left side or vice versa) advancing downstream. This phenomenon occurs due to the centrifugal force associated with the flow along a bend.

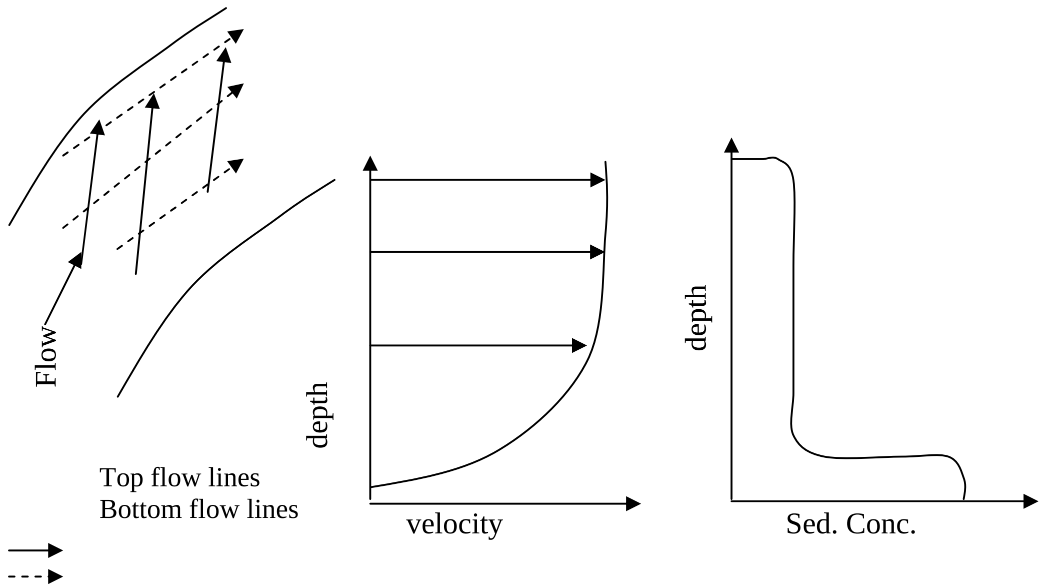


Figure 2.46: (a) Flow along a bend (b) Velocity Distribution Along Vertical (c) Sediment Conc. Along Vertical

From Figure 2.46, it is evident that the velocity v at the top is greater and that the top flow lines experience more centrifugal force F_C as $F_C \sim v$: Similarly, the bottom flow lines experience less centrifugal force. As a result, the flow lines separate in a three-dimensional space. The sediment deficient top flow lines, while diving down along the concave bank, pick up a large amount of sediments. Similarly, the bottom flow lines with a heavy sediment load move towards the convex bank and release the load there and then roll up again to reach the concave bank with relatively sediment free water. This phenomenon causes degradation of the bank toe, which ultimately makes the bank slope steeper than the critical one and slope failure takes place preceded by a tension crack on the land surface and the material (basal) is carried away by the river and the phenomenon is repeated at the new slope line. The bank retreats till the hydraulic and geo-technical conditions allow.

Revetments, Spurs and Cut-Offs

Revetments refer to an artificial surfacing of a riverbank and part of the riverbed in order to make the bank resistant to erosion. River spurs are structures constructed in the river channel in order to deflect the main flow away from an eroding bank. Cutoffs are short cuts across meanders, loops or new channels (diversions) that change the location of the main flow channel. Retards, although not common in Nepal, are structures that slow down the flow velocity and induce deposition thereby protecting a riverbank. Revetments and spurs are more appropriate in the mountain (or upper) river reaches while cutoffs and retards are more common in the lower river reaches.

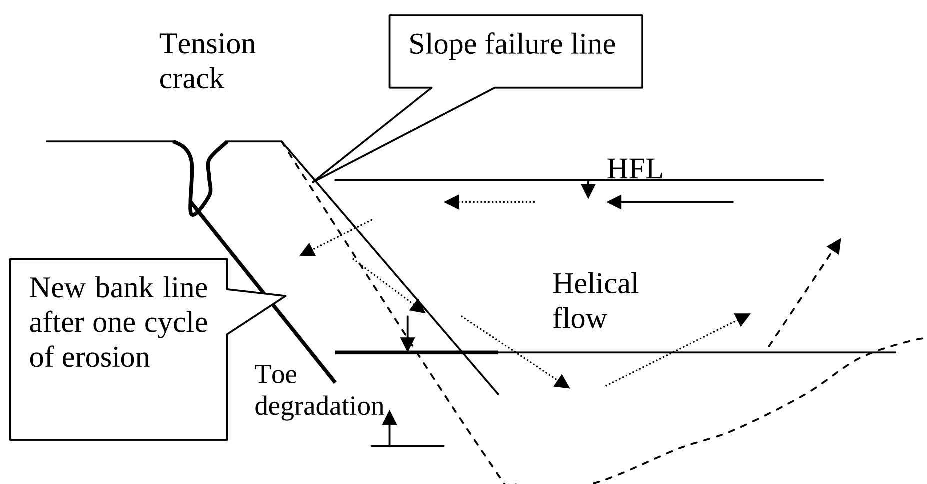


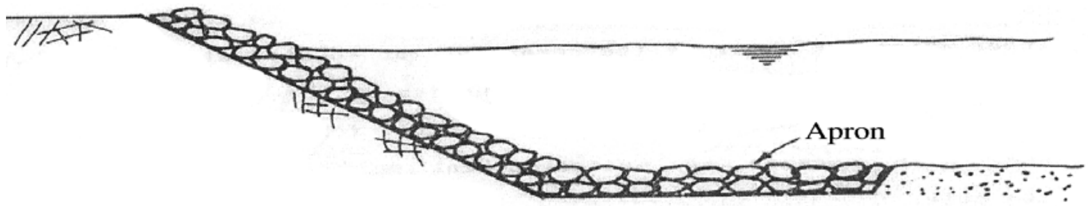
Figure 2.47: Typical bank erosion process

Revetments

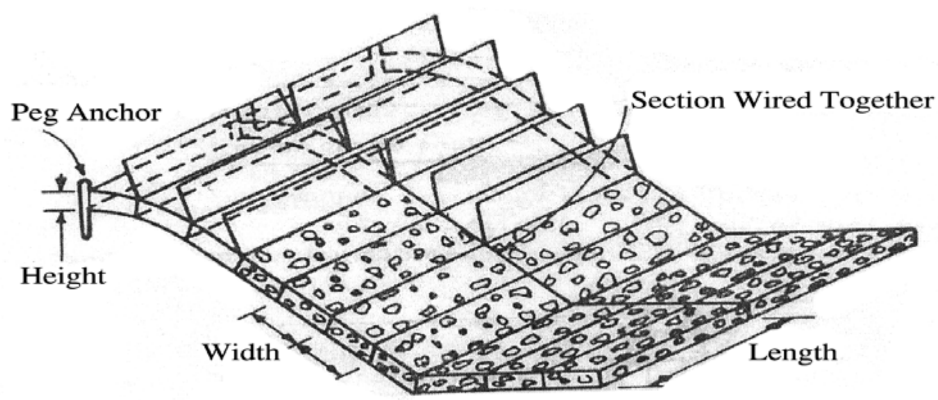
Revetments can be constructed from a variety of materials such as:

- Gabion's filled with stone,
- Stone (riprap),
- Timber piles,
- Bamboo piles,
- Old tires,
- Concrete slabs,
- Sand bags and
- Combinations of two or more of the above.

The selection of the most appropriate type of revetment depends primarily on the type of materials available near the construction site. In Nepal, the most common form of revetment is the gabion-stone basket and riprap (see Figure 2.48).



RIPRAP REVETMENT WITH APRON



GABION REVETMENT MATTRESS

Figure 2.48: Revetment of the gabion-stone basket and riprap

Spurs

Spurs are constructed into the flow channel and receive considerable "attack" at their ends or noses. Spurs serve to deflect flow away from an eroding bank and should generally be constructed as a series of spurs; three or more, since one spur may not adequately divert a flow channel to the other side of the flood plain. Spurs, as for revetments, can be made from a variety of materials and requires toe protection (apron) at their noses to protect against scour.

Retards

Retards are structures similar to spurs except that they are permeable, allowing water and sediment to flow through them. The structures reduce flow velocities resulting in deposition of suspended sediment eventually resulting in a natural build-up of the riverbank in front of the eroding bank. Retards can be constructed from a variety of materials. One of the more interesting types of retards is the "porcupine" which consists of bamboo boxes filled with brick and attached to long bamboo cross-members. These boxes are then wired together to form a retard field.

Guidelines for the design of retards

- The length of a retard line should correspond to the amount of an eroded river bank that is to be recovered by deposition.
- For velocity reduction along an eroding bank, retards should be placed in lines perpendicular to the bank with line spacing about equal to the length of the line.
- The top or the retard structure should generally slope down into the river instead of being horizontal.

Deflecting Panels

Top panels are used in increasing flow depth in a navigation river channel, whereas bottom panels, which are also known as submerged vanes or IOWA vanes, can be used to prevent bank erosion. The limitation of bottom panels is that they are effective in fine bed sediments (D_{50}<0.4 mm).

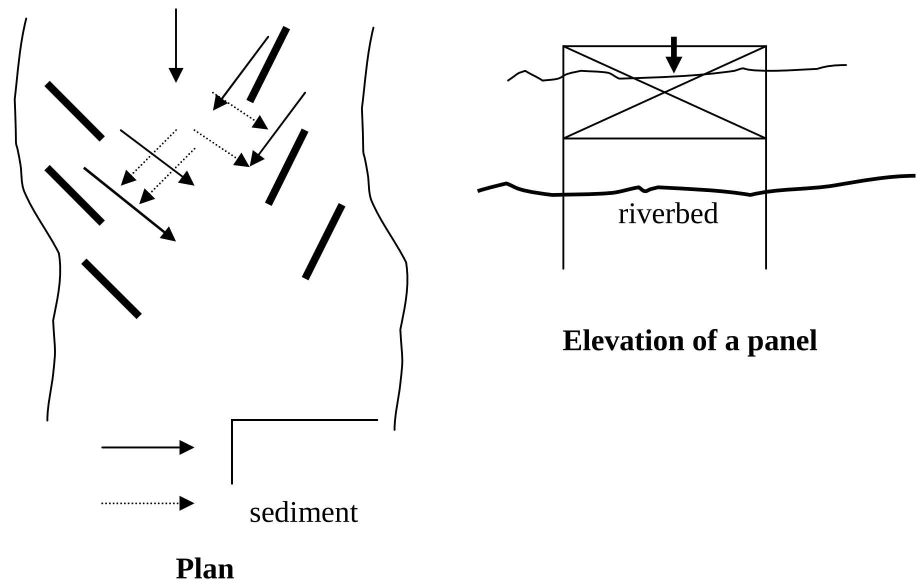


Figure 2.49: Layout of top panels

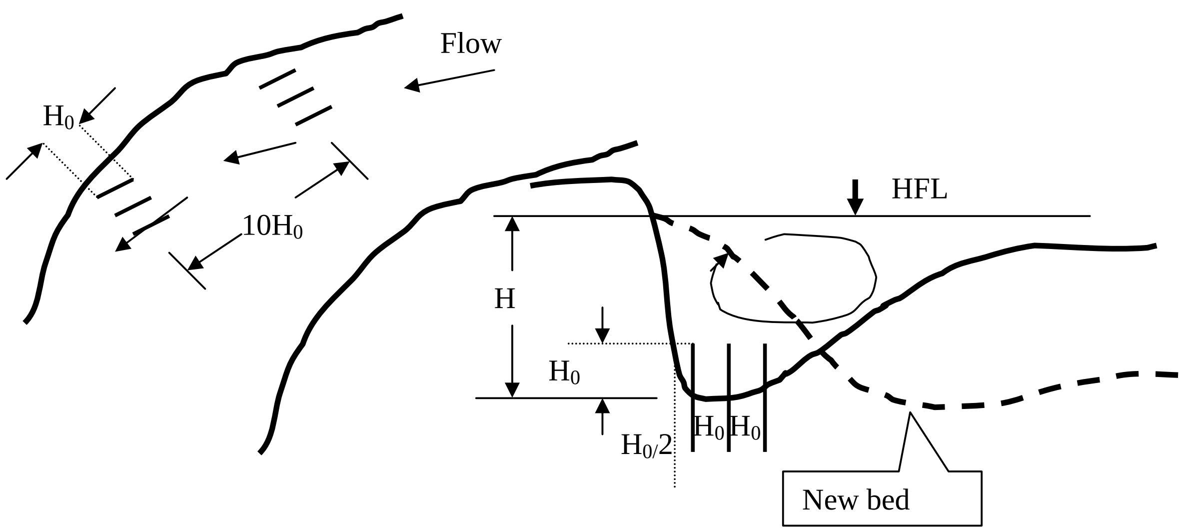


Figure 2.50: General arrangement of bottom panels

Pilot channels or cut-off channels are resorted to in case there are sharp bends in fine sand-bed rivers. At such bends, pilot channels are excavated for 15% to 30% of the dominant discharge to straighten the channel, which, due to the increased bed shear stress (due to steep slope), will widen naturally and the entire river takes a new course. After some flood events, the straightened channel again tends to assume its old course, thus protection measures may become necessary. Pilot channels often cause instability in the river equilibrium requiring continuous protection works to maintain the channel alignment.

Selective alternatives by checking predominant river characteristics

A.

- **Bank erosion at one location**
- **Width/depth ratio less than 50**
- **Irregular channel pattern**
- **Good flow alignment**

→ **Hold river in existing channel**

B.

- **Bank erosion at many locations**
- **Bed appears to be rising**
- **Many flow channels (split, braided)**
- **Width/depth ratio more than**
- **Poor alignment upstream**

→ **Relocate main flow channel**

C.

- **Bed distinctly being lowered, down cutting**
  Banks slumping over long length

↓

Hold vertical position of riverbed

Conclusion
A river makes its own regime for attaining equilibrium. Conventional theories regarding this are various regime theories. A river channel attains equilibrium and assumes its plan-form and profiles (cross section and long-section) whenever five extreme conditions are satisfied. The five extreme conditions are:

The river has a tendency to maximize

- Sediment transport rate
- Flow resistance
  and it has a tendency to minimize
- Stream power
- Unit stream power
- Bed slope.

The river keeps on changing its plan-form and profiles till these conflicting conditions are satisfied.

2.3 SELECTION OF THE BRIDGE AXIS

After the reconnaissance survey, an appropriate axis for the proposed bridge is selected. A surveyor has to make a preliminary design as per selected bridge axis to make the optimum site selection and also to define the investigation / survey area.

Preliminary Design

Make a sketch of the plan and cross section along the proposed axis in an appropriate scale. Mark all-important information. Preliminary design helps to define the survey area as well as technical feasibility of the site.

Preliminary design data for suspended bridge

Design parameter	SSTB standard	LSTB standard
Span	Up to 120m	Up to 350m
Permissible maximum Level difference, h	≤ l /25	≤ l /14
Preliminary sag, b	l/22	l/22
Appr. LP form higher side, f_{max}	l/14	l/15
For *l* > 120.0 m, Windguy layout	l/8 from axis both up and downstream	l/8 from axis both up and downstream

Preliminary design data for suspension

Design parameter	SSTB standard	LSTB standard
Span	Up to 120m	Up to 350m
Tower height, h_t	5.5, 7.35, 9.20, 11.05	0.145xl
Cable inclination, β_f	22°	25°
Backstay distance	Back distance from tower to Point at ground, touched by the line from tower top at angle, β_f	
Front of Tower block	About 3m from edge of 45° slope	About 5m from edge of 45° slope
For *l* > 120.0 m, Windguy layout	l/8 from axis both up and downstream	l/8 from axis both up and downstream

2.3.1 Selection of the Bridge Axis for SSTB Bridge

There are two possible standard bridge types namely "Suspension" and "Suspended". The selection of the bridge type mainly depends on the prevailing topography of the bridge site.

The Suspended type bridge is selected when the bridge foundations can be placed at a sufficiently high position giving the required free board from the highest flood level. The Suspension type bridge is selected when the bridge site is in comparatively flat terrain and the suspended bridge is not feasible due to the constraint of the free board. The Suspension type bridge is more expensive (per meter cost), and needs more inputs in design and construction than the Suspended type.

Therefore, assess the possibility of the Suspended type which should be the first choice. The Suspended type is more preferable due to economic reasons and easy construction technology which is more appropriate for the community bridge building approach. For further details on selection of the bridge type, refer to Chapter 4.10.2.

The selected site must be economically be justified and have a long life span and optimally serve the local people.

It must :

- fulfill the general condition
- have favorable river conditions
- have stable bank and slope conditions
- have the shortest possible span

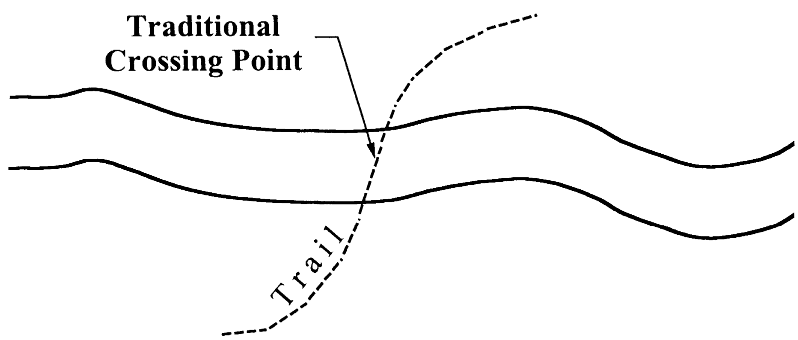
In case of SSTB bridge, a simplified method as described below is sufficient for bridge site selection

2.3.1.1 General Condition

The bridge site should fulfill the following general conditions. They are:

- close to traditional crossing point
- maximum bridge span of 120 m
- minimum free board from highest flood level
- safety distance for foundations

Use the following checklist to evaluate the general condition:

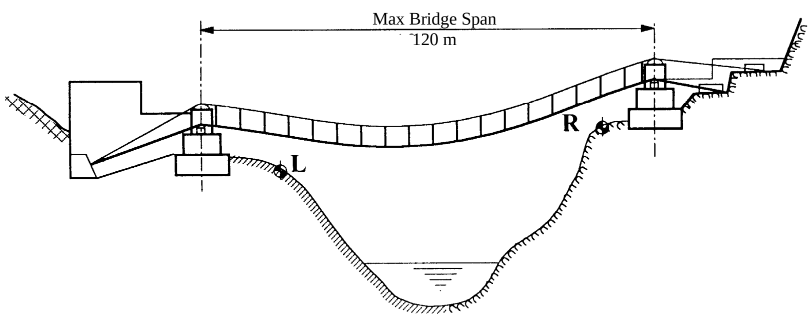
Features	***Condition***
TRADITIONAL CROSSING POINT The bridge site should be selected at or near the traditional crossing point.  • For a minor rivers, a detour from the traditional crossing point is not acceptable. • For major rivers, a detour up to 500 m u/s and 500 m d/s from the traditional crossing point may be acceptable.	**Favorable**: Selected site is at or near the traditional crossing point **Unfavorable:** Selected site is far from the traditional crossing point

<table>
<tr><th>Features</th><th>Condition</th></tr>
<tr><td>

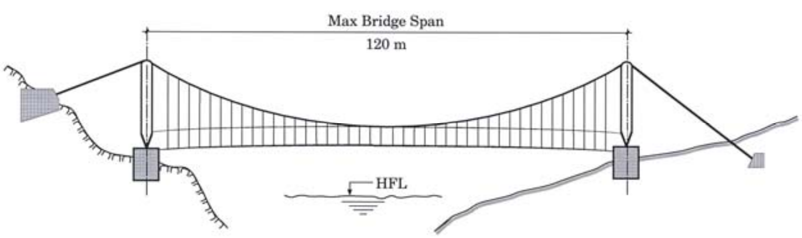
BRIDGE SPAN

The bridge span in this standard is limited to 120 m span.

In Case of Suspended Type Bridge:



In Case of Suspension Type Bridge:



- Measure tentative span
- Compare with the limit

</td><td>

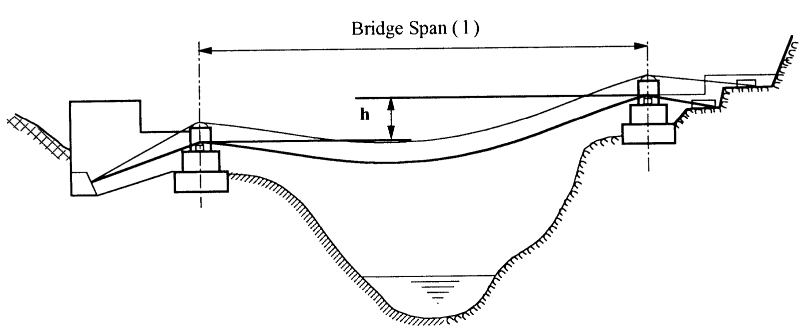
Favorable:
span, ℓ is equal to or shorter than 120 m

Unfavorable:
span, ℓ is longer than 120 m

</td></tr>
<tr><td>

LEVEL DIFFERENCE BETWEEN TWO BANKS

In Case of Suspended Type Bridge:
The level difference **h** between the two foundation blocks should not be more than ℓ**/25**.



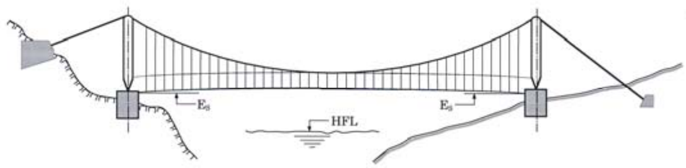
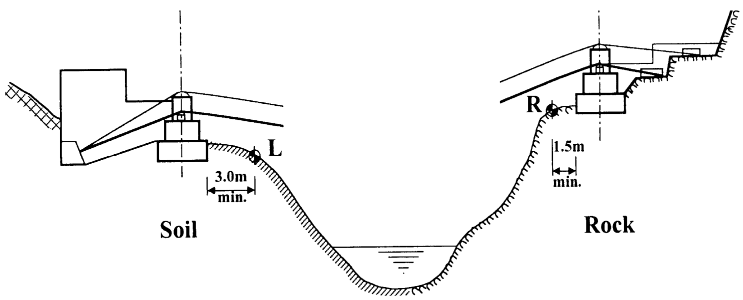
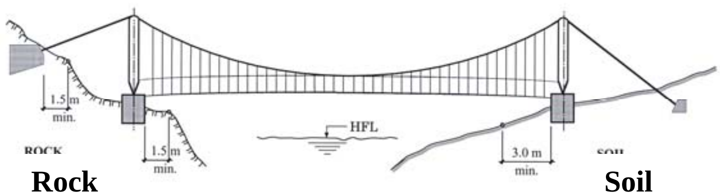
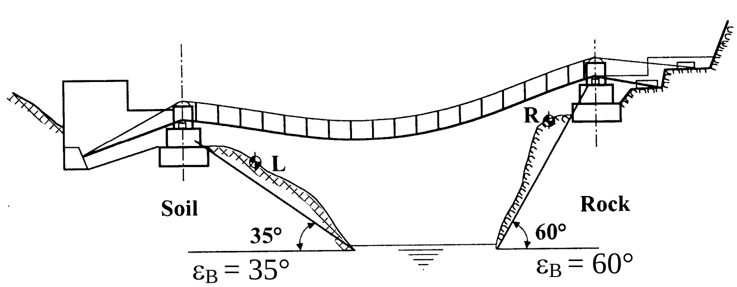
- Locate the tentative position of the bridge foundations on both banks
- Measure the level difference **h** between the foundations on the two banks
- Compare with the condition

</td><td>

Favorable:
h is equal to or less than ℓ/25

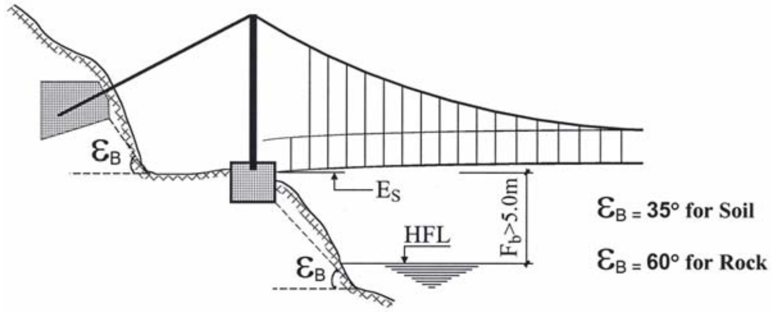
Unfavorable:
h is greater than ℓ/25

</td></tr>
</table>

Features	*Condition*
In Case of Suspension Type Bridge: The Walkway and Tower Foundations on both the banks should be placed at the same level.  • Locate the tentative position of the Walkway and Tower Foundations on the lower bank • Estimate the position of the Walkway and Tower Foundations on the other bank at the same level • Compare with the condition	**Favorable**: Walkway and Tower Foundations on both banks can be placed at the same level **Unfavorable**: Walkway and Tower Foundations on both banks can not be placed at the same level.
SAFETY DISTANCE FOR FOUNDATION The Foundations should be placed at least 3 m behind the soil slope and 1.5 m behind the rock slope from the front edge of the riverbank. **In Case of Suspended Type Bridge**:  **In Case of Suspension Type Bridge**: 	**Favorable:** Condition can be fulfilled **Unfavorable:** Condition cannot be fulfilled
SLOPE PROFILE The Bridge Foundation should be placed behind the line of the angle of internal friction. (The angle of internal friction is the angle of the slope of soil or rock at which it is still stable and does not slide). **In Case of Suspended Type Bridge**: 	**Favorable:** Condition can be fulfilled **Unfavorable:** Condition cannot be fulfilled

Features	***Condition***

In Case of Suspension Type Bridge:



- Draw slope lines of 35° (angle of internal friction) in case of a soil slope and 60° in case of a rock slope.
- The foundations should be placed behind these slope lines.
- Check if these conditions can be fulfilled.

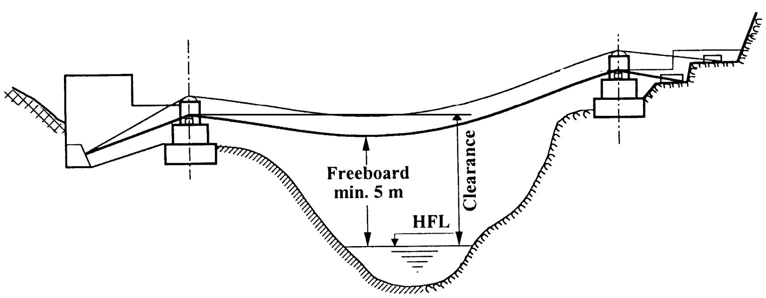
Favorable:
Condition can be fulfilled

Unfavorable:

Condition cannot be fulfilled

FREE BOARD

In case of Suspended Type Bridge:

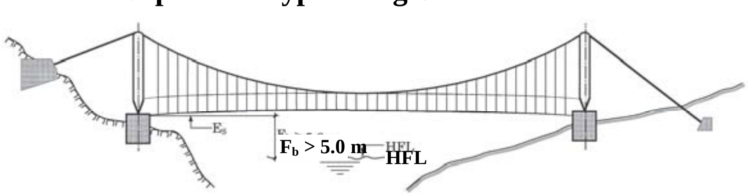


Favorable **Unfavorable**

The clearance between the lower foundation saddle and the HFL is:

Span	**not less than:**	**less than:**
up to 50 m	7.0 m	7.0 m
51 - 60 m	7.5 m	7.5 m
61 - 90 m	8.0 m	8.0 m
91 - 105 m	8.5 m	8.5 m
106 - 120 m	9.0 m	9.0 m

In case of Suspension Type Bridge:



- Identify the HFL by local observation and from villagers' information.
- Calculate the available clearance in case of the Suspended type or the free board in case of the Suspension type, and compare with the requirement.
- **Exception**: On flat or wide riverbanks the free board may be reduced. At gorges the free board may have to be increased.

Favorable:
Freeboard is not less than 5.0m

Unfavorable:
Freeboard is less than 5.0m

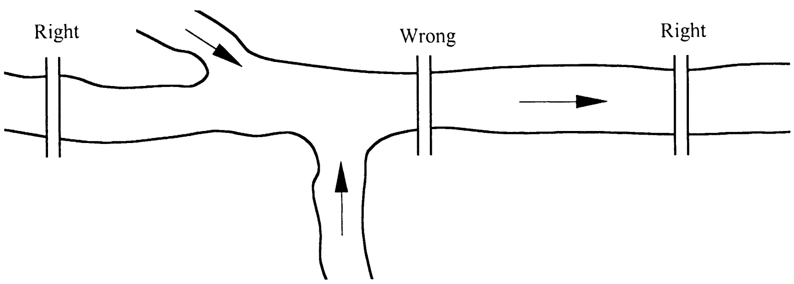
HFL = Highest Flood Level

2.3.1.2 River Condition

The selected bridge site must have favorable river conditions. Accordingly, a bridge should be located:

- on a straight reach of the river
- beyond the disturbing influence of larger tributaries
- on well defined banks

Use the following checklist to evaluate the river condition:

Features	***Condition***
RIVER FLOW In order to protect the bridge from sudden over-flooding and strong erosion, the bridge site should not be located near the confluence area of two rivers. 	**Favorable**: Bridge site far from river confluences **Unfavorable**: Bridge site near river confluences
RIVER BED The river bed at the selected bridge site should be stable without the possibility of erosion or filling up with bed load (boulders, gravel, silt, or sand)  River bed filled up after heavy flood	**Favorable**: River bed is not erosive, not filling up **Unfavorable**: River bed is erosive or filling up

2.3.1.3 Slope and Bank Condition

A bridge should be located at a site with safe and stable slope and bank conditions. The surveyor must identify any potential instability features or failure modes of the soil or rock slope and along the bank.

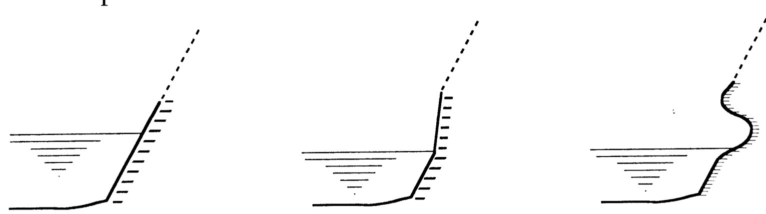
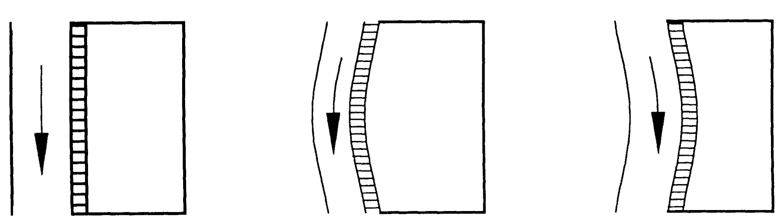
If the slope and bank is soil, potential instability features and failure modes are:

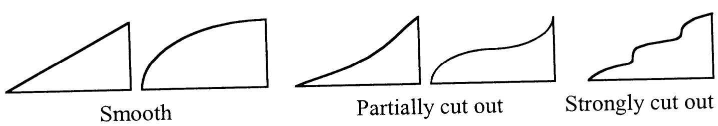
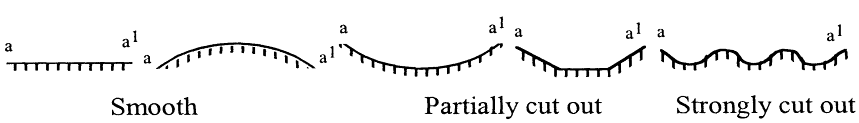
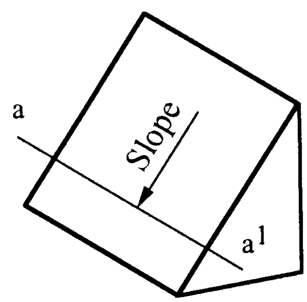
- bank erosion
- toppling instability of the bank
- erosion of the slope
- land slide

If the slope and bank is rock, the potential instability features and failure modes are:

- plain failures or a rockslide along the slope
- wedge failure leading to the fall of rock mass
- toppling leading to the fall of rock blocks
- translational Failure (slide) is similar to a landslide in a soil slope. Such failure is likely when the material of the rock is very weak (soft rock) and the rock mass is heavily jointed and broken into small pieces.

To avoid the above instability features, use the following checklist to evaluate the slope and bank of the selected site:

Features	***Condition***
IF THE RIVER BANK OR SLOPE IS SOIL **BANK PROFILE** The bank profile should be smooth.  Smooth Partially cut out Strongly cut out	**Favorable**: Bank profile is smooth to partially cut out **Unfavorable**: Bank profile is strongly cut out
RIVER BANK CONTOUR The bridge site should be located at a straight reach of the river to avoid undercutting or bank erosion.  Straight Convex Concave	**Favorable:** River contour is straight or convex **Unfavorable**: River contour is concave

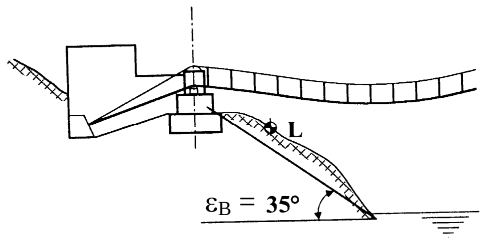
Features	***Condition***
BANK EROSION The river bank should not show any sign of erosion.  Bank erosion due to high river current	**Favorable**: No sign of fresh erosion **Unfavorable**: Presence of fresh erosion
SLOPE PROFILE The slope profile should be smooth.  	**Favorable:** Slope profile is smooth to partially cut out **Unfavorable**: Slope profile is strongly cut out
TRANSVERSE SLOPE The transverse slope should be smooth.  Transverse slope strongly cut out  	 **Favorable**: Transverse slope is smooth to partially cut out **Unfavorable**: Transverse slope is strongly cut out

<table>
<tr><th>Features</th><th>Condition</th></tr>
<tr><td>

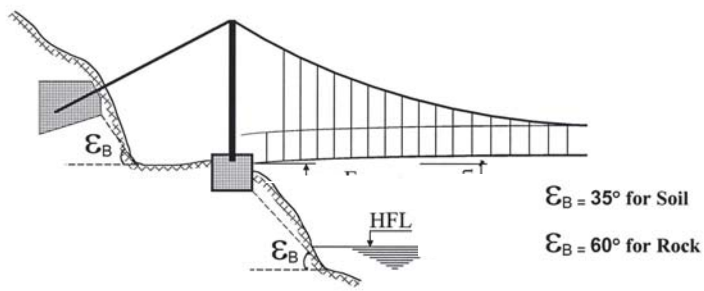
SLOPE INCLINATION (SOIL SLOPE)

The slope inclination should be less than 35°.

In case of Suspended Type Bridge:



In case of Suspension Type Bridge:



Estimate the slope inclination and compare it with the condition. If the site has an unfavorable slope inclination, it can still be selected provided the general condition of the slope profile is fulfilled.

</td><td>

Favorable:
Slope inclination is equal to or smaller than 35°

Unfavorable:
Slope inclination is greater than 35°

</td></tr>
<tr><td>

RIVER UNDERCUTTING

The bridge site should be free from river undercutting which may lead to landslide.

Landslide caused by river undercutting

</td><td>

Favorable:
There is no river undercutting

Unfavorable:
River undercutting is active or there is potential for river undercutting

</td></tr>
</table>

<table>
<tr><th>Features</th><th>Condition</th></tr>
<tr><td>INCLINED TREES

The selected site should not have inclined trees, which indicate an active landslide.

Inclined trees on landslide mass</td><td>Favorable:
Inclined trees are not present

Unfavorable:
Inclined trees are present</td></tr>
<tr><td>SEEPAGE OR SWAMPY AREA
The bank slope should not have any seepage or swampy area, which may lead to slope instability.</td><td>Favorable:
Seepage or swampy area is absent

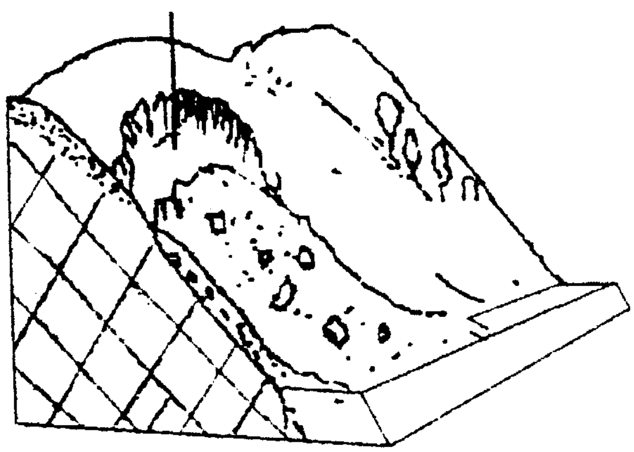
Unfavorable:
Seepage or swampy area is present</td></tr>
<tr><td>GULLY EROSION

No signs of gully erosion should exist within the vicinity of the selected site.


Active gully erosion

• Observe if any rivulets are within the vicinity of the selected site.
• If any rivulets exist, examine the dimension of the gully cutting.</td><td>Favorable:
No sign of gully erosion or only light gully erosion

Unfavorable:
Heavy gully erosion exists</td></tr>
</table>

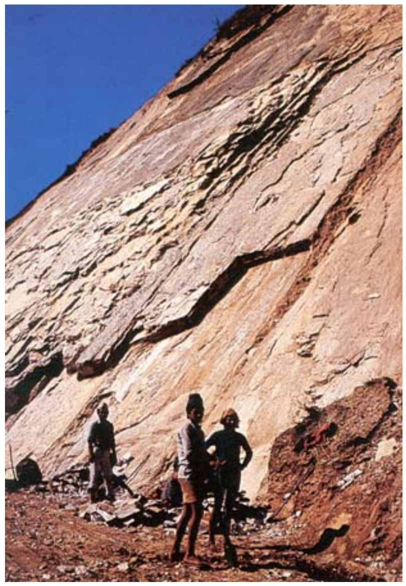
Features	*Condition*
SLIPPED (SLUMP) SOIL MASS The bridge should not be located on already slipped soil masses.  Slope with slipped soil mass **Back Scar**  **Slope Failure Model** • Examine and identify any indication of soil mass movement. This can be done by observing traces of back scars on the slope.	**Favorable**: There are no back scars or signs of soil mass movement **Unfavorable**: There are back scars or signs of soil mass movement

<table>
<tr><th>Features</th><th>Condition</th></tr>
<tr><td>

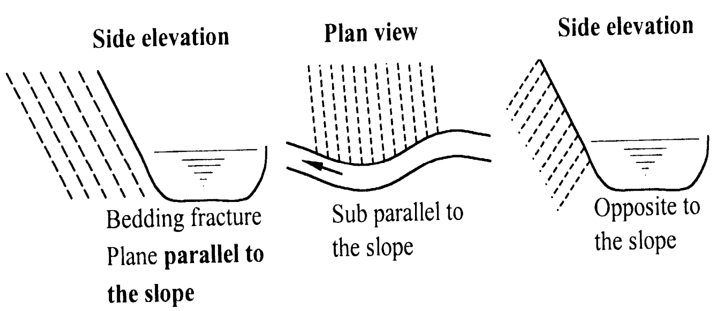
IF THE RIVER BANK OR SLOPE IS ROCK

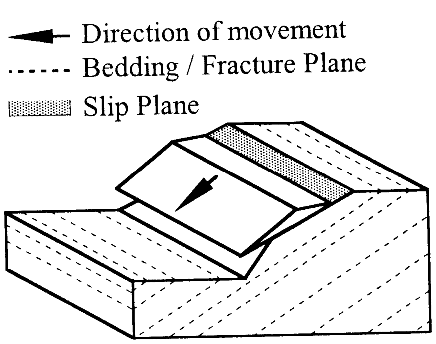
PLAIN FAILURE

Plain failures lead to the slide of rock layers along the slope. The rock bank/slope of the selected site should not have any feature of plain failure.



Bedding plain is **parallel to the slope** and plain failure is active.
Site is extremely unfavorable!





Plain Failure Model

- Identify bedding/fracture plain (layers of rock)
- Check its direction and inclination
- Compare with the condition

</td><td>

Favorable:

Plain failure will not take place, if:

- The bedding/fracture plain is sub-parallel or opposite to the slope
- The bedding/fracture plain is parallel to the slope, but inclination is less than 35^o

Unfavorable:

Plain failure will take place, if:

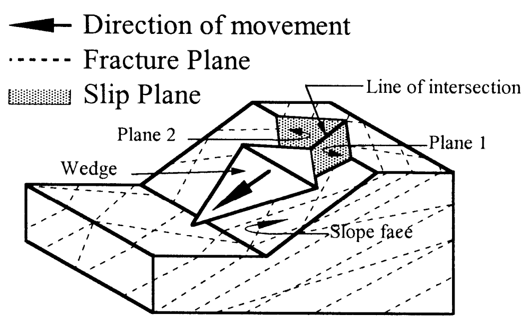
- The bedding/fracture plain is parallel to the slope and inclination is greater than 35^o
- There is presence of old rockslides

</td></tr>
</table>

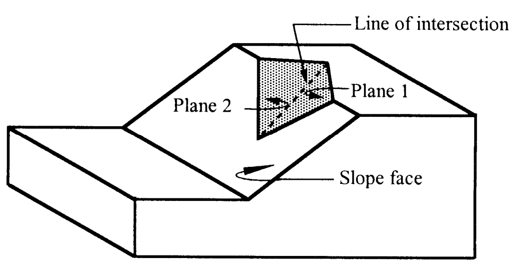
<table>
<tr><th>Features</th><th>Condition</th></tr>
<tr><td>

WEDGE FAILURE
Any form of wedge failure leads to sliding of rock masses. The rock bank/slope should not have wedge failures or potential wedge failures.

Traces of wedge failure



Wedge Failure Model



Wedge Failure

- Identify if there are fracture plains facing each other (intersecting)
- Check the inclination of the line of intersection
- Compare with the condition

</td><td>

Favorable:

Wedge failure will not take place, if

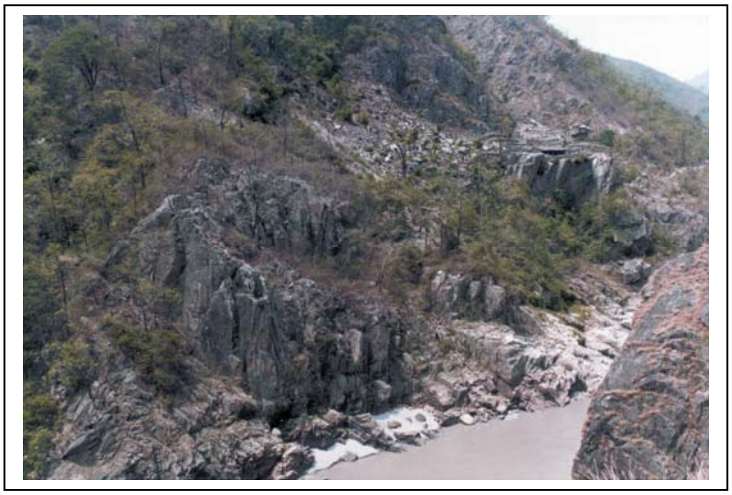
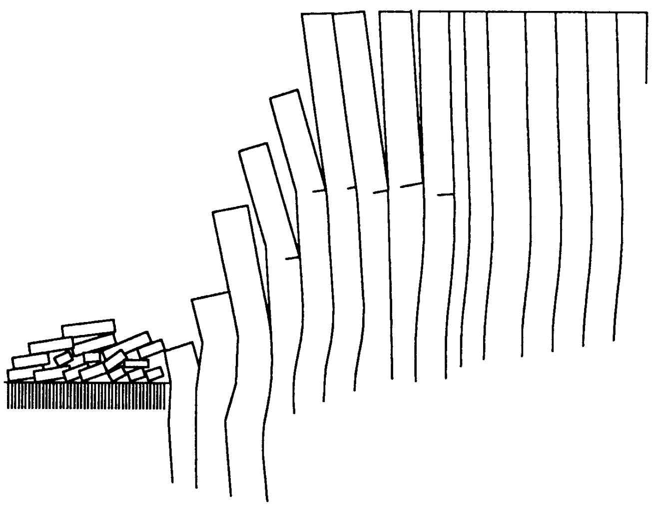
- There are no fracture plains facing each other
- There are two or more intersecting fracture plains but the inclination of the line of intersection is less than 35^o
- There are two or more intersecting fracture plains but the inclination of the line of intersection is opposite to the slope

Unfavorable:

Wedge failure will take place, if

- There are two or more intersecting fracture plains and the inclination of the line of intersection is greater than 35^o to the slope
- There is presence of wedge that has slid down

</td></tr>
</table>

<table>
<tr><th>Features</th><th>Condition</th></tr>
<tr><td>
TOPPLING FAILURE
The rock bank/slope should not have any features of toppling failure.

Potential toppling failure

Topling Failure model

Identify if there is a formation of vertically elongated rock blocks (cubes) due to vertical and horizontal fracture planes/joint
Estimate the inclination of the slopes
Estimate inclination and orientation of the rock block and compare with the conditions

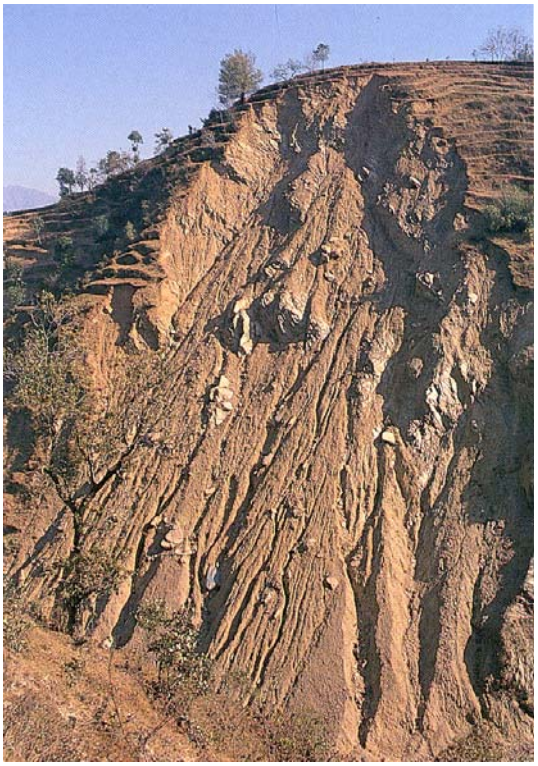
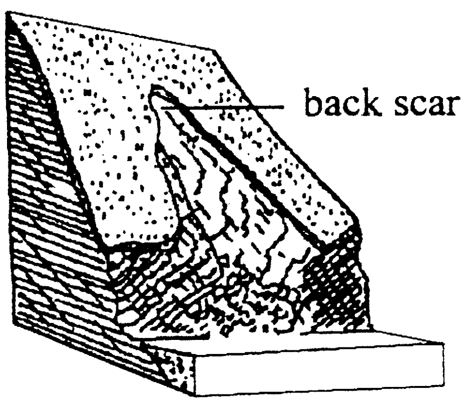
</td><td>
Favorable:
Toppling will not take place, if

The rock slope is less than 60^o
There is no formation of rock blocks
There is a formation of rock block but the b/h (width of the block/height of the block) is greater than 1

Unfavorable:
Toppling will take place, if

There is a formation of vertically elongated rock blocks in a steep slope greater than 60^o and the blocks are tilted towards the slope
Old toppled rock blocks are present

</td></tr>
</table>

Features	***Condition***
TRANSLATIONAL FAILURE (SLIDE) The rock slope should not have any potential of translational failure.  **Translational failure (sliding) of soft rock slope**  **Failure (slide) Model** • Identify the type of rock and its weathering grade • Estimate the inclination of the slope • Compare with the conditions	**Favorable**: **Sliding will not take place, if the** • Slope is hard rock • Slope is soft rock but not weathered • Slope is soft rock and weathered but not steeper than 40° **Unfavorable**: **Sliding will take place, if** • The slope is highly weathered soft rock and steeper than 40° • Back scars or old slides are present

2.3.1.4 Evaluation of the Bridge Site

After completing investigation of the site as per Chapter 2.3.1.1 to 2.3.1.3, categorize the bridge site as

Good	All or most of the features are favorable, and the surveyor is confident about the stability of the slopes, proceed with further survey work.
Bad	Most of the features are unfavorable. Reject the site.
Questionable	Most of the features are favorable and some are unfavorable. The site is questionable. In this case, further detailed investigation by an experienced geo-technical engineer is necessary. For detail refer to Chapter 2.3.2.

As far as possible, the bridge site should be selected at a location where protection works will not be required. If protection works are unavoidable, determine the required special structures like retaining wall, drainage channels, etc. A tentative design with dimensions and location of these structures should be illustrated in a sketch showing a plan view and a typical section. But it is best to **avoid bridge sites which require river protection works.**

2.3.1.5 Classification of Soil and Rock

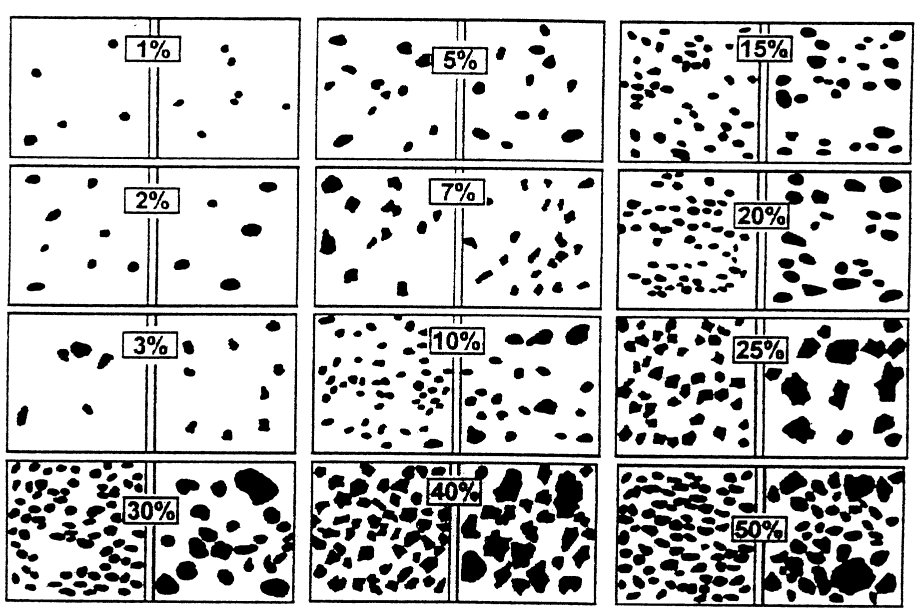
Identification of Soil and Rock types is required for appropriate foundation design. Soil and Rocks can broadly be classified as per the following tables.

Soil Classification

<table>
<tr><th colspan="2" rowspan="2">Soil Type</th><th rowspan="2">How to Identify</th><th colspan="3">Soil Parameters</th><th colspan="2">Applicable Main Cable Anchor Block Design</th></tr>
<tr><th>Bearing Capacity, [kN/m²]</th><th>Angle of Internal Friction, φ°</th><th>Unit Weight, γ [kN/m³]</th><th>For Suspended Type</th><th>For Suspension Type</th></tr>
<tr><td rowspan="2">Coarse Grained Soils
More than half of the materials are of individual grains visible to the naked eye (grain size bigger than 0.06 mm)</td><td>Gravelly Soils</td><td>Estimate the percentage (%) of coarse grains larger than 6 mm.
If more than half of the coarse fraction is larger than 6 mm, the soil is Gravelly Soil</td><td>400-600
(400)</td><td>32-38
(35)</td><td>19</td><td rowspan="4">Deadman Anchor</td><td rowspan="4">Deadman Anchor
or
Gravity Anchor Block</td></tr>
<tr><td>Sandy Soils</td><td>If more than half of the coarse fraction is smaller than 6 mm grain size, the soil is Sandy Soil</td><td>200-300
(200)</td><td>31-37
(33)</td><td>18</td></tr>
<tr><td rowspan="2">Fine Grained Soils
More than half of the materials are individual grains not visible to the naked eye (grain size smaller than 0.06mm)</td><td>Silty Soils</td><td>Prepare moist soil ball from the soil sample and cut it with a knife.
If the cut surface is dull or scratched, the soil is Silty Soil</td><td>150-200
(150)</td><td>30-32
(30)</td><td>17</td></tr>
<tr><td>Clay</td><td>Prepare moist soil ball from the soil sample and cut it with a knife.
If the cut surface is smooth and shiny, the soil is Clay.</td><td>100-200
(100)</td><td>9-25
(22)</td><td>16</td></tr>
</table>

For estimating the percentage (%) of coarse grains, use the following figure:

Ratio of coarse grains



Rock Classification

Rock Type	Examples	How to Identify	Degree of Fractures or Weathering	How to Identify	Rock Parameters		Applicable Main Cable Anchor Design	
					Bearing Capacity [kN/m^2]	Angle of Sliding Friction φ°	For Suspended Type	For Suspension Type
Hard Rock	Quartzite Limestone Granite, Dolomite etc.	Gives metallic sound at hammer blow	Rock is sound and fresh to fairly weathered	Rock has no sign of weathering or only faint signs of weathering up to 1-5 cm thickness	1500 - 2000 **(1500)**	35-50 **(40)**	**Drum Anchor in Hard Rock**	**Drum Anchor in Hard Rock or Gravity Anchor Block**
			Highly fractured rock and fresh to fairly weathered	In the rock shows widely open cracks, fractures and bedding	1500	35-50 **(40)**	**Drum Anchor in Fractured Rock**	**Drum Anchor in Fractured Rock or Gravity Anchor Block**
Soft Rock	Phylite Slate Siltstone Claystone Schist etc.	Gives dull sound at hammer blow	Fresh	No sign of weathering	1300	25-40 **(30)**		
			Fairly to highly weathered	Most of the original rock has been seriously altered Rock can be broken by hand	600-750 **(650)**	25-40 **(30)**	**Deadman Anchor**	**Gravity Anchor Block**

2.3.2 Selection of the Bridge Axis for LSTB Bridge

Bridge site Selection for LSTB Bridge is more demanding. The surveyor should have in-depth geo-technical knowledge for this.

2.3.2.1 Preliminary Study for Alternative Sites

Different alternative sites have to be studied. Among the alternative sites, best site is selected for the bridge axis. Use the following checklist for preliminary study of the alternative sites.

Checklist No. 1: Preliminary study for alternative sites

Bridge Number :__________________ Name :______________________

Alternative Site No :__________________ Approx Span :_________________

Approx. distance from the existing traditional Crossing point :________________

S.N	Description	Comment	R/B	L/B	Comment	R/B	L/B
1.	Riverbank Erosion	Present			Absent		
2.	River Current	Striking Bank			Straight		
3.	Vegetation	Heavy			Light		
4.	Landslides	Present			Absent		
5.	Slope Type	Soil			Rock		
6.	Steepness: Soil	$<35^0$			$>35^0$		
7.	Steepness: Rock	$<50^0$			$>50^0$		
8.	Seepage	Present			Absent		
9.	Springs	Present			Absent		
10.	Swampy Area	Present			Absent		
11.	Erosions	Present			Absent		
12.	Inclined Trees	Present			Absent		
13.	Rivulets	Present			Absent		
14.	Cliff/s (Soil / Rock)	Present			Absent		
15.	Others (Specify)						

On basis of the conditions filled up in the checklist, the surveyor has to make his own judgment and evaluate the status of the bridge site. It has to be stated whether the banks are good, fair or questionable for each bank.

2.3.3 Slope and River Bank Condition

Use the following checklist for the slope and river bank condition study.

CHECK LIST NO. 2: SLOPE STUDY & SITE SELECTION

Bridge Number : __________	Name : __________
Bank : __________	Approx. span : __________
Azimuth of bridge axis : __________	Azimuth of river flow : __________
Slope type : __________	__________
Approx. distance from the traditional existing	crossing point : __________

This Check List has to be filled in after selecting the best site by using Check List No. 1.

A) SLOPE AND RIVER BANK DESCRIPTION

General aspect of the slope:

Smooth	☐	Partially cut-out	☐
Cut-out	☐	Strongly cut-out	☐

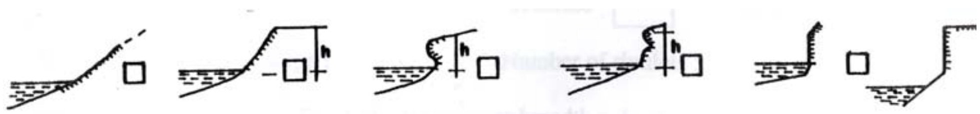
Average inclination and dimensions of:

River bank	:	________	°	Slope	:	________	°
Height of bank	:	________	m	Length of slope	:	________	m
Breadth of slope	:	________	m				

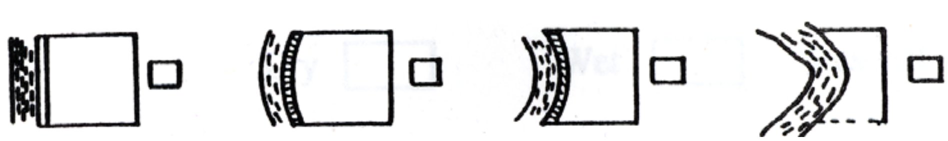
General slope profile :

Shape of transverse section of the slope :

General river bank profile :



Contour of river bank:



Rock out crops or scarps on slope and bank:

Slope :	Sparse ☐	Moderate ☐	Numerous ☐
Bank :	Sparse ☐	Moderate ☐	Numerous ☐

Vegetation cover on the slope: Heavy ☐ Few ☐ Moderate ☐ None ☐

Deforestation: Heavy ☐ Light ☐ Moderate ☐ None ☐

Paddy field: Location : ________________ Present ☐ Absent ☐

Irrigation channel: Location : ________________ Present ☐ Absent ☐

B) RIVER

Flow type: Perennial ☐ Calm ☐ Non perennial ☐ Turbulent ☐

Fordability: Fordable ☐ Fordable in dry season ☐ Non-fordable ☐

Erosiveness: Highly erosive ☐ Non-erosive ☐ Moderately erosive ☐ Filling up ☐

C) INSTABILITY FEATURES

Bank erosion : Heavy ☐ Light ☐ Moderate ☐ None ☐

Gully erosion: Heavy ☐ Light ☐ Moderate ☐ None ☐

Sheet erosion: Heavy ☐ Few ☐ Moderate ☐ None ☐

Water run off on the slope : Number of rivulets : _____________

Give approx. Dimension of each rivulet, average breadth x depth

1. _____ x _____ 2. _____ x _____ 3. _____ x _____ 4. _____ x _____

Dry ☐ Wet ☐ Seasonal ☐

Presence of impermeable layers : Absent

Present ☐ Azimuth __________ Distance _________ from A or B,

Absent ☐

Presence of swampy area: Absent ☐ Present ☐

Seepage: Absent ☐ Permanent ☐ Seasonal ☐

Springs : Absent ☐ Permanent ☐ Seasonal ☐

Presence of inclined trees : Absent ☐ Present ☐

Bulges or depression :

Present ☐ Azimuth __________ Distance _________ from A or B,

Absent ☐

Transverse open cracks: Present ☐ Absent ☐

Longitudinal open cracks: Present ☐ Absent ☐

Traces of dissolution on slope & bank: Few ☐ Numerous ☐

Fallen blocks or rock-fall on slope and bank:

Absent ☐ Few Numerous ☐ ☐

Angular ☐ Medium Angular ☐ Rounded ☐

Max. diameter ___________ m Location _________________________

Landslides : Absent ☐ Dormant Absent ☐ ☐

Failure mechanism (if present):

Erosion ☐ Plane ☐ Translational ☐

Rotational ☐ Flow ☐

Wedge ☐ Fall ☐

Landslides or fallen debris :

Present ☐ Azimuth __________ Distance _________ from A or B,

Absent ☐

Old slided wedge: Absent ☐ Low ☐

High ☐ Numerous ☐

Density of geologic planes: Low ☐ Moderate ☐

High ☐ Not visible ☐

Opening of geologic planes : Closed ☐ Open ☐

Very open ☐ Not applicable ☐

Dip of bedding plane :

Parallel to the slope ☐ Sub parallel ☐ Opposite ☐

Weathering of rock : Sound ☐ Fair ☐ High ☐

Judgment of Bank	**Action to be taken**
Good	Proceed with further investigation
Acceptable	Proceed with further investigation Propose protective measures
Questionable	Proceed with further investigation Propose protective measures Consult Engineer Geologist
Unstable	Choose a new site