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GOVERNMENT OF INDIA
MINISTRY OF RAILWAYS

GUIDELINES FOR CUTTINGS IN RAILWAY FORMATIONS

Guidelines No. GE: G-2

April - 2005

GEO-TECHNICAL ENGINEERING DIRECTORATE
RESEARCH DESIGNS AND STANDARDS ORGANISATION
LUCKNOW-226011

PREFACE

Indian Railways are going through a major technological upsurge. There is a large scale upgradation and renewal going on all round. New tracks are coming up in hitherto uncharted territories. Majority of the new alignments will pass through cut sections rather than embankments. Naturally, this is posing more challenges than ever before. Experience shows that the best maintained cut sections are the best constructed ones and vice-versa. This calls for providing optimum technical inputs at the construction stage itself. Good engineering practice demands that adequate attention is paid at every stage- survey, alignment selection, in understanding geology of terrain, analysis of data and design, apart from adhering to standards of construction and maintenance.

Railway formations in cuttings have unique problems as they pass through hilly terrain having different characteristics in regard to type and deposition of soil. Moreover, failures in cuttings take place normally without advance warning causing risk to safe running of traffic. So far, there are no elaborate instructions dealing with cuttings on Indian Railways. Keeping this in view, a draft on 'Guidelines on cuttings' was prepared by RDSO & submitted to Railway Board in July, 2003. Thereafter, Railway Board had nominated a committee of following officers for review of draft prepared by RDSO vide letter no. 2003/CE/1/MB/2 dated 30.01.04.

Name	**Designation**
Shri Ghansham Bansal	Professor IRICEN / Pune
Shri S. N. Maske	Dy. CE/Con/CR
Shri P. V. Reddy	Sr. DEN(C)/PGT/SR
Shri S. V. Mahalank	Dy. CE(Geologist)/KRCL
Shri A. K. Chaturvedi	Sr. DEN/SER/GRC
Shri A. K. Singh	Director/GE/RDSO
Shri Nand Kishore	Executive Director/GE/RDSO

After holding five meetings, committee finalized the draft at its level in November 2004. Vide letter no. 2002/CE-II/PRA/15/CRS dated 01.11.04; Railway Board had further instructed RDSO to get the draft technically scrutinized by outside agency. As a follow up, Geological Survey of India (GSI), Lucknow and CRRI, New Delhi were requested to offer their valuable comments on the draft finalised by committee.

After receiving comments of GSI & CRRI, committee again met on 7th & 8th April'05 at RDSO to deliberate upon the suggestions given by GSI and CRRI & finalised the present draft. The chapter on "Laterite Cuttings" has been written by Shri G.Narayanan, CE(C), S.Railway. During deliberation and preparation of this booklet, valuable assistance was rendered by Shri R. K. Premi, SRE/GE/RDSO.

This booklet is designed to educate the field engineers about the likely problems of cut sections and the methodology to arrive at the best possible solution within the economical means at their disposal. It also lays down the guidelines for construction as well as maintenance of cuttings.

The book draws upon various other guidelines and manuals to present the subject comprehensively. A lot of illustrations and field studies have been included to make the subject relevant to the everyday needs of the construction and maintenance railway engineers.

(Nand Kishore)
Exe.Director,GE,RDSO,

April, 2005
RDSO, Lucknow

FOREWORD

The expansion of railway network in the country arising out of the need to connect remote and farflung areas with the main land, has led to construction of new lines in hilly terrain and consequently, through cuttings. This development underlined the importance of subject of cuttings and necessitated the issuance of comprehensive guidelines on them.

I am pleased to note that RDSO has brought out the guidelines on cuttings at the right juncture. The draft as prepared by RDSO has been reviewed by a committee specially constituted for this purpose. This draft has been scrutinized by GSI and CRRI and their comments/suggestions have been incorporated to enhance its value. The guidelines deals with the subject of cuttings adequately and has also taken into account the various practices and experience of other railway systems.

I am sure that the guidelines will prove to be useful to all railway engineers and will go a long way in meeting out the need for elaborate and consolidated instructions on the subject. Your feedback may be sent directly to Executive Director (GE), RDSO or Executive Director (P), Railway Board, who will be glad to take care of your suggestions.

Member Engineering
Railway Board, New Delhi.

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CHAPTER I

INTRODUCTION

1.0 The need to cut into natural ground arises from the fact that railway formations have to maintain relatively easier gradients than the prevailing ground slopes. Railway cuttings for BG single line are generally 6.25 m wide with side drains on both sides, and a suitable side slope. A cutting is a permanently open excavation. The cut profile (or cutting) may pass through soils, rocks or a combination of both. The nature of the soil/rock through which it is made governs the parameters of design such as the slope of the sides, drainage; need for retaining walls, method and rate of excavation and to a great extent its cost. Hence, a prior knowledge of the subject and a detailed survey of the project area, relating the findings of the study to the knowledge base regarding the behaviour of soils/rocks will be immensely helpful in arriving at the right engineering judgment.

2.0 SOIL

Soil is the uppermost layer of the crust on earth. It is formed very slowly by the weathering of rocks and contains a mixture of mineral matter and organic matter, such as decayed leaves, flowers, minute bacteria etc.

2.1 Soil Classification

2.1.1 Transported Soils

These soils are made up of particles which have been derived from a parent rock of distant location and have been deposited at its present place by wind or water action. These soils are generally homogeneous and easy to test for their properties. Hence, the behaviour is predictable and treatment /modification to suit the needs is easier.

2.1.2 Residual Soils

These soils continue to stay over their parent rock where they were formed from the in-situ weathering (physical or chemical) and decomposition of rock and have not changed the position ever since. Residual soils can have characteristics that are quite different from those of transported soils. Particles of residual soil usually consist of aggregates or crystals of weathered mineral matter that break down and become progressively finer if the soil is manipulated. The permeability of residual soils may not be related to its own granulometry (e.g. by the well known Hazen formula) like in case of transported soils; instead it is usually governed by its micro and macro fabric, jointing and by super imposed features such as termite and other bio-channels present in the soil.

TABLE 1.1: CLASSIFICATION OF WEATHERED ROCK MASS (AS PER ISRM)

Sl. No.	Symbol	Degree of weathering (%)	Term	Description
1.	W_0	0	Fresh	No visible sign of material weathering.
2.	W_1	Less than 25	Slightly	Discolouration indicates weathering of rock on major discontinuity surfaces.
3.	W_2	25 - 50	Moderately	Less than half the rock material is decomposed and or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.
4.	W_3	50 - 75	Highly	More than half the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.
5.	W_4	Over 75	Completely	Majority of rock material is decomposed and or disintegrated to soil. The original structure of the rock mass is still intact.
6.	W_5	100	Residual soil	All material decomposed. No trace of rock structure preserved.

In the brief, the main characteristics of residual soils are:

- Very heterogeneous, which makes sampling and testing for representative parameters difficult.
- Usually highly permeable, therefore, susceptible to rapid changes in material properties when subjected to changes of external hydraulic condition.

In general, the process of formation of a residual soil profile is complex and very difficult to model and generalise. Therefore, simplified weathering profiles which contain material of different grades are usually the basis to describe the degree of weathering and the extent to which the original structure of the rock mass is destroyed indicated by the affected depth from the ground surface. The weathering profile is an important information for undertaking slope stability analysis. It also controls the following processes-

- The potential failure surface and the mode of failure.
- The groundwater hydrology, and therefore the critical pore pressure distribution in the slope.
- The erosion characteristics of the materials.

2.1.2.1 <u>Laterization and Special Considerations</u>

Lateritic soils constitute an important group of residual soils of India, covering an area of around 100,000 sq. km. They are derived from the rock called ‘laterite’. Laterites are formed by the decomposition of the rock, removal of silica and bases due to excessive leaching on account of high humidity and rainfall and deposition of aluminium and iron. Generally, a coarse-grained concretionary material with 90% or more of this material is known as 'Laterite', while relatively fine grained material with lower concentrations of oxides is referred to as 'Lateritic soils'. As such, the problems associated with Laterites arise basically due to exposure of Lateritic soils in cuttings. Some typical problems and their solutions have been discussed later in Chapter IV.

3.0 <u>ROCKS</u>

The earth is made up of a number of layers like an onion. The density of the material making the earth increases as we go towards the centre. The outermost layer is called the earth’s crust and is composed of the lightest of the earth-materials. The crust has two parts. The upper part is known as *sial*. It is rich in silica and alumina. Below the *sial* are the layers of rocks known as *sima*. This layer consists mainly of silica, iron and magnesium. Both the *sial* and *sima* layers together form the crust of the earth having an average thickness of about 60 km. Below the crust of the earth lies the mantle having a thickness of about 2840 km. The core is the heaviest part as it is made up of heavy metals, mainly iron and nickel. Its thickness is about 3467 km.

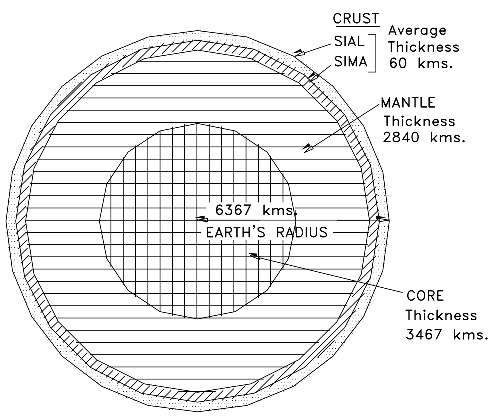


FIG. 1.1 INTERIOR OF THE EARTH

Rock is natural accumulation of mineral matter in earth’s crust or upper mantle consisting of an aggregate of mineral particles. A mineral, as the knew, may be defined as an inorganic substance with consistent physical properties and a fixed chemical composition. With the exception of some forms of carbon forms, sulphur and a few metals, all minerals are chemical compounds, each containing two or more elements in fixed proportions by weight. The way in which the composition of the earth's crust is dominated by eight elements is shown in Table 1.2 below.

TABLE 1.2

Elements in the Earth's crust		Approximate Proportion (%)
Oxygen	(O)	46.7
Silicon	(Si)	27.7
Aluminium	(Al)	8.1
Iron	(Fe)	5.0
Calcium	(Ca)	3.6
Sodium	(Na)	2.8
Potassium	(K)	2.6
Magnesium	(Mg)	2.1
	Total	**98.6**

These elements comprise approximately 98% of the earth's crust and together with other elements form twelve common minerals (Table 1.3) which make up 99% of all rocks in the earth's crust. The remainder of the known rock-forming minerals, numbering over 1000, make up less than 1% of the earth's crust. Most rocks will consist of two or more of these minerals, each of which has a particular set of physical properties which may affect the engineering properties of the rock as a whole.

TABLE 1.3

Mineral	Hardness (Moh Scale 1-10)	Specific Gravity
Felspars		
(i) Orthoclase	6	2.6
(ii) Plagioclase	6	2.7
Quartz	7	2.65
Micas		
(i) Muscovite	2.5	2.8
(ii) Biotite	3	3
Amphiboles		
Hornblende	5-6	3.05
Pyroxenes		
Augite	5-6	3.05
Olivine	6-7	3.5
Calcite	3	2.7
Dolomite	4	2.8
Kaolinite	1	2.6
Haematite	6	5

3.1 Classification of Rocks

Generally speaking, rocks are classified as hard and soft rocks. Hard rocks are those excavation of which involves intensive drilling and blasting. They can stand vertical or even over-hanging cut depending on type/mass and dip of the rock. Soft rocks are those which can be excavated by crow bars and/or pick axes or mechanical excavators normally without use of blasting. However, classifications of rocks are as varied as their properties. They are based on their origin, chemical composition, strength, state of decomposition, looks, or physical properties. It may be required to follow more than one method to understand the rock properties in greater detail.

3.1.1 Geographical Classification

Igneous

These are formed by solidification of molten magma, either at depth in the earth's crust or by extrusion. They are classified as plutonic, hypabyssal, or volcanic, depending on the depth and rate of their cooling with its effect on their texture or crystal size. When the magma intrudes or thrusts under existing rocks, it cools slowly and the minerals in it are able to form big crystals. But when the magma forces its way to the surface, it cools rapidly and minerals in it either form very small crystals or none at all. Examples of igneous rocks are granite and basalt.

Sedimentary

Such type of rocks are formed by the deposition (usually under water) of products largely formed by the destruction of pre-existing igneous rocks. Forces of nature are constantly at work, wearing down the rocks of earth's crust. The particles or fragments of rocks are carried away by moving ice, running water and wind to be deposited in layers under water or even on land. In time, the grains are compacted together by pressure due to their own weight and that of sea water from above. They are cemented together by minerals from sea water. Dead bodies of marine animals are also deposited on the bed generation after generation. In time, they get pressed into layers and become sedimentary rocks. These rocks tend to be weaker than igneous rocks, largely because of the hydration of felspars to form kaolinite and the introduction of organic minerals such as calcite.

From an engineering point of view, the three main groups of sedimentary rocks are–arenaceous (sand) rocks, argillaceous (clay) rocks and calcareous (limestone) rocks. Typical arenaceous rocks such as sandstone, quartzite or breccia consist of discrete fragments of mineral-usually quartz, held together by a matrix of calcite. An argillaceous rock such as a clay or shale consists of minute particles held together weakly and comprising largely kaolinite. The calcareous rocks consist of organic remains or precipitates, mainly in the form of calcite (limestone).

Metamorphic

Metamorphic rocks are formed from either igneous or sedimentary rocks which have altered physically and sometimes chemically by the application of great heat or high pressure at sometime in their geological history. This may happen when land subsides or hot lavas burst out from below. Under the influence of pressure or heat, or both acting together, the mineral composition, or texture or both of the original rock are changed forming new layers of rocks. Examples are gneiss, marble, quartzite, slate etc. Intense heat changes limestone into marble and sandstone into quartzite. Slate results from action of pressure on shale. Gneiss is formed from granite and other igneous rocks.

3.1.2 Compositional Classification

Based on main constitutional element, rocks can be classified as-

Agrillaceous
Main constituent is alumina and examples are slate, laterite.

Silicious
Main constituent is silica and examples are quartzites and granite.

Calcareous
Main constituent is lime and examples are marble and lime stone.

3.1.3 Physical Classification

Stratified
Such types of rocks shows distinct layers along which it can be easily split in to thin slabs.

Unstratified
These types of rocks show no sign of stratification and can not be easily split into thin layers.

3.1.4 Rock Strength Classification

This system is based on uniaxial compressive strength of rock, which is defined as maximum applied load at failure divided the cross sectional area of rock specimen.

TABLE 1.4

UNCONFINED COMPRESSIVE STRENGTH (Mpa)	ROCK QUALITY
> 100	Very Strong
50 – 100	Strong
12.5 - 50	Moderate Strong
5.0 – 12.5	Moderately Weak
1.25 – 5.0	Weak
0.6 –1.25	Very Weak Rock or Hard Soil
0.3 – 0.6	Very Stiff Soil

0.15 – 0.3	Stiff Soil
0.08 – 0.15	Firm Soil
0.04 – 0.08	Soft Soil
< 0.04	Very Soft Soil

3.1.5 Classification Based on Density & Porosity

Dry density is defined as density of rock in a dry state when all moisture has dried out of voids. Porosity is defined as the ratio of pore volume to the bulk volume.

TABLE 1.5

DRY DENSITY(t/m³)	ROCK QUALITY	POROSITY (%)	ROCK QUALITY
<1.8	Very Low	>30	Very Low
1.8 – 2.2	Low	30 - 15	Low
2.2 – 2.55	Moderate	15 - 5	Medium
2.55 – 2.75	High	5 – 1.0	High
>2.75	Very High	<1.0	Very High

3.1.6 Rock Quality Designation (RQD)

This system is used for the quantitative description of rock quality in drill cores and serves as a valuable tool for the assessment of the jointed condition of rock masses. It is based on analysis of recovered core taking into account the number of fractures and the amount of alteration or softening in the rock mass as observed in rock cores from a bore hole. Instead of counting of fractures, the total length of core pieces of size equal to and longer than 10 cm, which are hard, and sound is measured and the same is taken as a %age of total core length. A good judgment is necessary in case of sedimentary rocks and foliated metamorphic rocks to differentiate between fresh and existing fractures and this method is not as exact as in igneous and thick massive bedded deposits. The limitation of RQD system is that it disregards the influence of joint tightness, orientation, continuity and the presence of gouge material. The correct procedure for measurement of the length of core pieces and calculation of RQD are summarized in figure below-

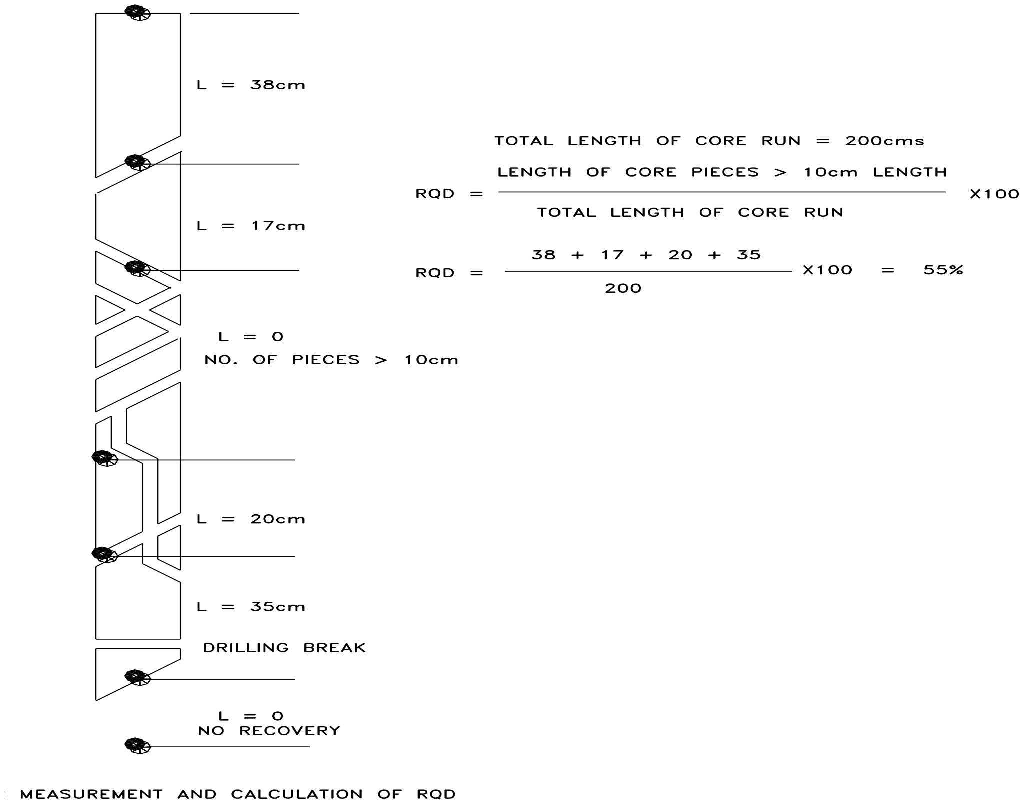


FIG 1.2 : MEASUREMENT OF RQD

3.1.7 Rock Mass Rating (RMR)

In this system, the following six parameters are used in rock classification –

1. Uniaxial compressive strength of rock material
2. Drill core quality RQD
3. Spacing of joints
4. Condition of joints
5. Groundwater conditions
6. Orientation of joints

TABLE 1.6

RMR (Rock Mass Rating) (IS: 12070-1987)	**Rock Quality**
0 – 20	Very poor
21 – 40	Poor
41 – 60	Fair
61 – 80	Good
81 – 100	Excellent

3.2 Typical Tests For Rock Samples

Tests for soils are generally well understood by the Railway engineers. The following tests for rock properties are to be performed for assessment of rock quality for design purposes.

TABLE 1.4: TESTS ON ROCKS AND RELEVANT STANDARDS

Procedure	*ASTM Method/ Code	BIS Code	Comments
Specific gravity and absorption of - Coarse aggregate - Fine aggregate	C – 127 C – 128	2386(Pt.3)	Helpful in stability analysis
Soundness of aggregate by use of Sodium Sulphate or Magnesium Sulphate	C – 88	2386(Pt.5)	To indicate rock's resistance to weathering
Resistance to degradation by impact and abrasion using the Los Angeles machine of - Large size coarse aggregate - Small size coarse aggregate	C – 535 C – 131	2386(Pt.4)	To serve as a measure of degradation of mineral aggregates from a combination of actions including abrasion or attrition, impact and grinding.
Examination of aggregates by petrography	C – 295	2386(Pt.8)	Helpful in understanding the nature of rock & useful in stability analysis
Compression characteristics - Uniaxial - Triaxial strength of undrained rock core specimen without pore pressure measurement	SPECIAL D – 2664	IS:9143-1979 IS:13049-1991	To classify rock for strength and deformation properties. Utilizes diamond drilling cores To find angle of shearing resistance of weak rock material with random orientation of joints. Range of normal stresses occurring in field are applied.

*ASTM - American Society for Testing Materials
(US organization for setting various standards)

3.3 Commonly Found Rocks

3.3.1 Basalt: It is a fine grained hard rock formed when the magma comes to the surface and cools rapidly. In terms of mechanical strength, basalt is one of the most competent of common rocks. Formed by volcanic action, it has a micro-fine texture and consists of micro-crystals of augite and plagioclase held together by strong mechanical bonding. A feature of basalt and other extrusive igneous rocks which may sometimes reduce its strength is the presence of voids formed by trapped gases unable to escape during its rapid cooling. Its uniaxial compressive strength varies from 61 MPa to 355 MPa where as bulk density and porosity are 2.8-2.9 gm/cc and 0.1-1.0 % respectively.

3.3.2 Granite: It is a hard rock formed by slow cooling under the crust. It is also strong, but its coarse texture and the presence of large crystals of orthoclase in particular, tend to make it substantially weaker than the fine-grained igneous rock, and more comparable with the harder sandstones. Granite is formed by plutonic cooling of magma and , in these conditions, the orthoclase felspar, with a rather higher melting point than the other major constituent, quartz, tends to crystallize out surrounded by polycrystalline quartz. In other words, the sedimentary structure of mineral particles in a matrix is simulated, and the strength of the granite depends to a certain extent on the presence of other minerals such as mica tending to weaken the 'matrix'. There is a marked absence of voids. Its uniaxial compressive strength varies from 12 MPa to 300 MPa and the bulk density and porosity are 2.6-2.7 gm/cc and 0.5-1.5% respectively.

3.3.3 Sandstone: It is a typical sedimentary rock consisting of rounded quartz particles cemented together by a calcite, ferruginous or silica matrix. The strength depends mainly on the strength of matrix and the type and amount of pore space on it. A sedimentary quartzite (silica matrix) will be stronger than granite. A large-grain calcareous sandstone weakly cemented with a high proportion of non-contact void spaces will be extremely weak. Its uniaxial compressive strength varies from 40 MPa to 179 MPa. Typical values of bulk density and porosity are 2.0-2.6 gm/cc and 5-25 %respectively.

3.3.4 Shale (mudstone): It is a compressed clay consisting of micro-fine particles of kaolin, mica and quartz, normally in the micron range. A cemented shale can approximate in character to a concrete or weak sandstone. Shales differ from clays in that compaction gives the clay material a certain molecular cohesion which is not entirely lost under wet conditions. A shale, on the other hand, loses all strength when wet, failure depending solely on density and load; similar effects may occur in compacted shales where mechanical disturbance may lead to reversion, but the presence of 'slippery' minerals such as kaolinite.

The weakness of shales is primarily due to their relative lack of compaction and hence high pore space. If subject to higher pressures this can be reduced, with consequent increase in strength, in the form of mudstones and eventually slates. The presence of large proportions of fine-grain quartz (silt-stone) will also increase strength.

All varieties of these shales, without exception, disintegrate very quickly and easily when exposed to the atmosphere. Nearly all shales have a great capacity for absorbing moisture. This property may create endless trouble as, on absorbing water, they swell and then shatter. This often causes enormous landslides.

Some of the dark coloured shales have deliquescent salts such as sodium chloride, sodium sulphate and potassium sulphate. These salts absorb water and moisture and have properties of breaking the shales into clay. During rains, these salts are readily dissolved and the mass, lubricated with clay, flows like bituminous material. Typical values of bulk density and porosity are 2.0-2.4 gm/cc and 10-30 % respectively.

3.4 Igneous rocks are with few exceptions competent, massive and strong while sedimentary rocks are weak and strongly foliated and jointed. Of these sedimentary rocks, the arenaceous and calcareous, under favourable conditions, approach nearest to the ideal of the igneous rocks. The argillaceous rocks depart furthest from them. The earth's crust is made up of 95% igneous rocks, 5% sedimentary rocks and an insignificant proportion of metamorphic rocks. This does not, however, give a completely true picture of the rocks likely to be encountered by engineers working in rock. The earth's crust may be assumed to be varying from 30 to 50 km in thickness and virtually all major works take place in the top few kilometers which contain the major part of the sedimentary rocks. This means that the engineer working on the earth's surface or in near-surface mineral deposits must contend with rocks which are often sedimentary, and also with some metamorphosed rock.

3.5 Glossary of Terms

For better appreciation of the subject, few terms of engineering geology, as relevant to Railway engineers, are given below in alphabetical order-

BEDDING: Bedding is the surface parallel to the plane of deposition. Applies to rock resulting from consolidation of sediments and exhibiting surfaces of separation (bedding planes) between layers of same or different material e.g. shale, sandstone, limestone, etc.

BENCHING: It is the formation of a series of level platforms upon a slope.

BOULDER: It is rock fragment with diameter minimum plan dimension greater than 300 mm and weight not less than 40 kg.

CLEAVAGE: Tendency to cleave or split along definite parallel plane which may be highly inclined to the bedding.

CUT AND FILL:
It is a term used to describe any section of earthwork which is partly in cutting and partly in filling.

CATCH WATER DRAIN:

It is a drain excavated on the upper slope of a hill area to intercept and collect water flowing towards the formation, and to lead it to a point where no damage will result to the formation or in general, it is a drain to catch water flowing to a certain area and drain it off to another area.

CATCHMENT AREA:

It is the area from which the rainfall flows into a drainage channel at any specified section.

CUTTING: An excavation that allows railway lines to pass through surrounding ground at an acceptable level and gradient. The term excludes track ballast, blanketing, formation, geosynthetics directly associated with track performance capping layers.

DISCONTINUITIES:

These are structural features which separate intact rock blocks within a rock mass. Joints, faults & fractures are all discontinuities. These discontinuities are planes of weakness along which there is little or no tensile strength. Slope failures depend on the extent, type & pattern of discontinuities in the rock mass.

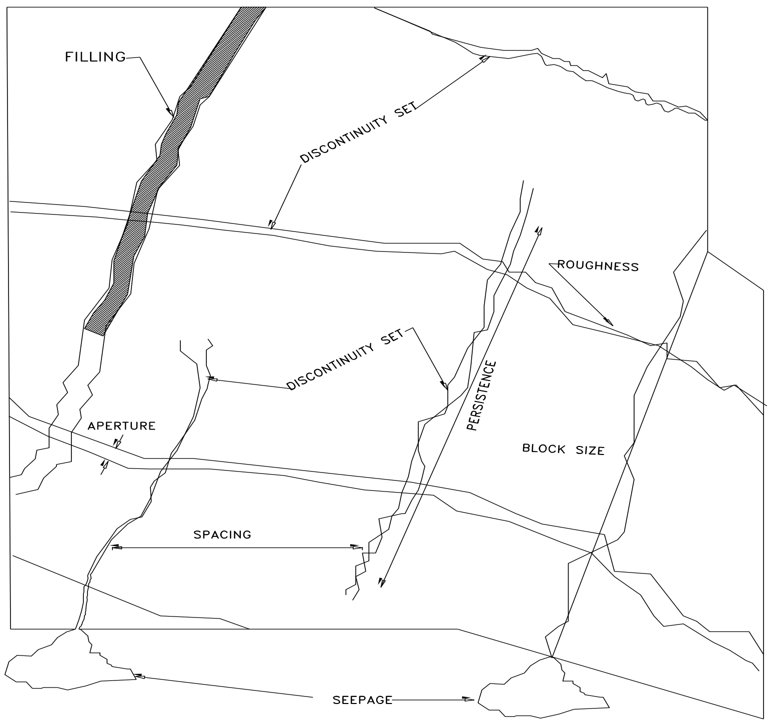


FIG.1.3 DESCRIPTION OF DISCONTINUITIES

DIP: The angle at which a strata or other planar feature is inclined from horizontal. Basically, it is the angle with which rock is dipping against horizontal plane. It is given as an amount (dip angle) and direction as shown in the diagram below.

DRAIN: It is a conduit or channel, either artificial or natural, for carrying off surplus ground water or surface water.

EROSION: It is the process of removal of matter from the banks of a stream or other surfaces by the action of natural forces like flowing water, wind etc.

FAULT: A fracture or fracture zone along which there has been displacement of two sides relative to one another parallel to the fracture. To put it simply, faults are rock fractures along which the opposing blocks of rock have moved or moving.

FOLD: A bend or deformation in the strata or other planar structure within a rock mass. Folds are formed when the rocks are plastically deformed resulting in anticlines and synclines.

GOUGE: This is infilling material between two faces of a structural discontinuity such as fault, shear plane.

JOINT: Joints are fractures in rocks along which there is no displacement of the adjacent blocks parallel to the plane of the fracture.

LITHOMARGIC CLAYS:
That part of Lateritic formations that lies just above the base rock, where the process of Laterization is not complete.

MORPHOLOGY:
Study of processes causing changes(to the earth's surface).

NATURAL SLOPE:
Sloping ground that has been formed by natural processes.

ROCKFALL: A fall of rock off a cliff or very steep outcrop.

ROCK CUTTING:
A cutting where discontinuity set characteristics are conducive to planar, wedge, toppling or raveling failure.

STRIKE: It is the direction of a line formed by the intersection of the bedding planes of starta or foliation planes with the horizontal.

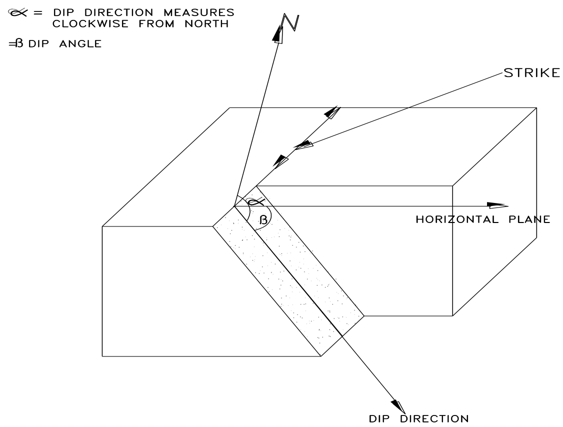


FIG.1.4 DIP AND STRIKE OF BEDDING

SIDE DRAIN:

It is a drain along the side of a railway formation.

SUB SURFACE DRAIN:

It is a drain below the ground surface to drain away sub-soil water

SLOPE FAILURE:

Any significant rock fall or soil slip, slide, or flow in a cutting or natural slope, or significant soil slide or slip in an Embankment or Natural Slope.

TEXTURE: The arrangement in space of the components of a rock body and of boundaries between these components.

THIXOTROPY:

The property of liquefying on being shaken and of reforming on standing.

TOE WALL:

It is small retaining wall structure at the foot of an earth slope.

VALLEY: It is an elongated low land between ranges of mountains or hills often having a river or stream running along the bottom.

WEEP HOLE:

It is a small opening left through soil retaining structure to drain away percolated water.

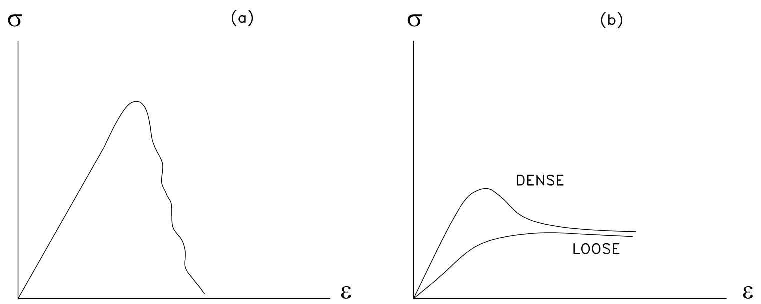
WEATHERING:

It is the breakdown of rocks at the earth's surface as a result of chemical reaction or mechanical abrasion.

CHAPTER II

CUTTING FAILURE

1.0 Like any other solids, soils and rocks have limited resistance to stresses induced by external loads or by their own weight. This limited resistance may result in collapse or failure of a part of the soil/rock mass if the induced stresses reach some critical value. The actual mode of failure depends on the mechanical behaviour of soil or rock, on the state of stress, and constraints imposed by the surrounding non-failing mass. Violent failure, termed 'brittle' which is associated with kinetic energy release, often occurs in rocks, whereas failure of soils, termed as 'yielding', is a relatively gentle process. The brittle failure is manifested by rapid drop in stresses beyond the peak strength. Much less pronounced drop in stress is observed in dense soils, with the soils ultimately reaching the residual strength. The yielding of loose soils is not preceded by the drop in stress. Figure given below shows typical behaviour of rocks and soils under compression



STRESS–STRAIN RELATIONSHIP. (a) FOR ROCKS (b) FOR SOILS

Difference in the behaviour at failure results primarily from the different order of magnitude of the strength of internal bonds in the material. Rocks have very strong inter crystalline or inter particle bonds, whereas soils are slightly cemented or not cemented at all. The essential feature of both materials, however, is the frictional nature of their resistance to deformation in addition to the resistance of bonds. The frictional resistance is well pronounced by the formation of distinct local or global slip surfaces, along which the material may become discontinuous.

2.0 Failure of cuttings is largely understood as failure of slopes. However, occasional obstructions caused to the movement of traffic by isolated drops of debris along the slope and flooding of cuttings during rainstorms are also avoidable failures. Therefore, failure of cuttings can be discussed under the following broad groups.

a) Erosion failure
b) Flooding, and
c) Slope failure

2.1 Erosion Failure

Erosion of cuttings by wind or water action ultimately results in failure, as explained below.

i) **Wind**: Wind erosion is a prominent feature of day-today existence in arid zones, where sand dunes shifting due to the action of wind block the track. There is a certain predictability about their movement and it is possible to undertake evasive action.

ii) **Water**: Erosive action of water on slopes and adjoining area results in movement of soil material leading to deposition of same on the formation. Design of adequate catch water drains and vegetation cover on cutting as well as catchments area can be very effective. Geojute, geo-netting may be used wherever feasible to check erosion due to action of water.

2.2 Flooding Failure

Failure by flooding is often caused in new cuttings that have not stabilized yet. Flooding of formation floor by either water or loose silt/clay deposited by water is very commonly seen. Either of the cause is avoidable, if adequate drainage plan is prepared and executed during construction. Erosion of cut slope at toe is dangerous from structural stability point of view also.

2.3 Slope Failure

Slope failure of cuttings is a complex phenomenon. The failure of rock slopes is controlled by geological features such as bedding planes and joints which divide the rock body up into a discontinuous mass. Under these conditions, the failure path in rocks is normally defined by one or more of the discontinuities. However, in the case of soil, a strongly defined structural pattern does not exist and therefore, the failure surface is free to find the path of least resistance through the slope.

2.3.1 Stability of Slopes

Soils

Failure in soils is rather simple i.e. circular in homogeneous materials & non-circular or planar in layered soils. In general, failure along a non-circular surface can be anticipated if the soil deposit is non-homogeneous or if there are discontinuities within the slope. A predominantly planar slip surface may be expected in shallow natural slopes. Failure generally takes place along slip surfaces parallel to the slope.

The stability of any slope is governed by a number of factors such as the nature of materials comprising the slope, the history of slope formation, the movement of water through the soil and the steepness of the slope. The most common type of failure in soil is that due to sliding and it is often referred to as a shear failure along a surface of sliding. The tendency for instability or failure is a consequence of gravity (self-weight of the soil or soils comprising the slope) and any other external loads (e.g. a structure on the crest of the slope, water pressure in tension cracks or an earth tremor). This tendency is resisted by the shear strength of the soil or soils comprising the slope. In a stable slope there is no continuous surface along which the average shear strength is less than the average shear stress caused by gravitational and external loads. Zones of overstress could be the starting points for local, partial or complete failure. The formation and propagation of such zones is sometimes crucial for the safety of a soil mass. To understand the conditions governing stability fully, it is useful to consider geological, geotechnical and environmental factors.

The soil adjacent to the ground and slope surfaces may be quite strong but there may be a bedding plane or a fault or an ancient surface of sliding with in the slope. An understanding of local geology facilitates possible detection of such features at a given site.

The shear strength that can be mobilized is governed by the permeability characteristics and the extent of drainage and volume change that can take place. Such geotechnical factors require careful attention. Infiltration of water due to rainfall increases pore water pressure and reduces the shear strength. Often slope failures occur as a consequence of heavy or prolonged rainfall.

Environmental changes near a sloping area such as deforestation, urbanization and construction of reservoirs often lead to increases in pore water pressure and other effects such as soil erosion. Filling of valleys may also disturb the natural drainage characteristics of a sloping area and contribute to instability.

Soil slopes may be classified into two main categories, viz. natural slopes and man-made slopes.

a) A natural slope which is stable and has not been disturbed in its recent past represents a long-term condition of equilibrium under changing environmental conditions. The balance between disturbing forces (DF) and resisting forces (RF) may change and with it the safety margin (SM) which maybe defined as follows:

   SM = (RF - DF)

   The stability of a slope may be endangered by external disturbance and by the infiltration of water from different sources such as rainfall or melting snow. If the safety margin is still positive, the slope will survive otherwise failure will take place. If failure does not take place, the safety margin will increase again after the influence of external disturbance ceases to exist. Changes in the safety margin of a stable slope can not be observed directly and may only be inferred on the basis of theory and through indirect observation. Many natural slopes are only marginally stable and may show visible signs of impending failure after disturbance such as excavation, abnormal loading, earth tremor and infiltration of water. The sudden formation of tension cracks at the crest and bulging of the toe of a slope may indicate the likelihood of complete collapse.

In general, a very slow decrease in stability of all natural slopes takes place which should not be confused with the relatively sudden fluctuations due to disturbance or infiltration of water. It is , however, difficult to estimate the rate of this very gradual decrease with any degree of certainty or accuracy. Accordingly the time, when a natural slope which has been stable for many years might reach critical equilibrium, can not be predicted. However, if doubts about stability exist at a given site on the basis of the performance of similar slopes in the surrounding area, an observational programme may be undertaken to monitor the slope. Adoption of such a procedure has to be justified in terms of economics, technical feasibility and the requirements of time. In many situations, it may be more economical to take preventive or remedial measures. The decision must be based on inference drawn from observation and from the analysis of available data concerning similar slopes in the surrounding area.

b) Man-made slopes are those which form the sides of embankments, earth dams, earth-rockfill dams, excavations and spoil-heaps.

The stability of an excavation depends on the properties of the soil forming the sides and base of the excavation, the slope inclination and the slope height. The process of excavation represents unloading and the manner in which excavation is carried out may have considerable influence on stability. While embankment construction is a loading process which increases normal stresses with in the ground, excavation decreases normal stresses. However, both processes, result in a change in the magnitude and orientation of shear stresses. This change is an adverse one for stability and its severity depends on the slope height, slope inclination as well as the method and sequence of construction.

2.3.1.1 Distinction must be made between short-term and long-term stability conditions, especially for slopes of cohesive soil. In the field, the end of construction situation is usually a short-term stability condition. The long-term condition is when ‘excess’ or ‘transient’ pore water pressures within a slope are fully dissipated. However, the long-term condition of equilibrium may be reached in the field after many months or years depending on the thickness of cohesive soil, its coefficient of permeability and other factors. For cuts, excavations and natural slopes, critical stability is in the long term when the factor of safety is a minimum.

Unloading or excavation causes negative excess pore water pressure. Consequently, the total pore water pressure has its lowest value at the end of construction and shear strength has its highest value at the time. In the long term, the negative excess pore water pressure reduces to zero, the total pore water pressure is increased and the shear strength is, therefore, decreased. It is obvious that the stability of an excavation is reduced in the long term from the condition at the end of construction.

The complete dissipation of excess pore water pressures in a cohesive soil may take many years. In cohesionless soils like sand, excess pore water pressures are dissipated so rapidly that there is no need to distinguish between short-term and long-term conditions on the basis of pore water pressure and drainage. However, this is true of static loading only. Significant excess pore water pressures may develop in such soils during earthquakes resulting in dramatic loss of shear strength.

Rocks

The properties of intact rock are changed dramatically by the presence of discontinuities such as joints, faults and fractures. These discontinuities are planes of weakness across which there is little or no tensile strength. In essence, discontinuities break the cohesive bonds across distinct planes in the rock. On a local scale, this may cause the tensile strength to drop to zero and will usually cause significant reductions in shear strength as well as large increases in permeability. On a regional scale, discontinuities are largely responsible for the distinctive drainage patterns and major erosional features that are characteristic of faulted and jointed terrain.

In rocks, most slope failures are controlled by ever present discontinuities. Slope failures will propagate depending on the extent, pattern and types of discontinuities present in the rock mass. The orientation of these discontinuities in combination with the natural face of rock, shall bring about one or more failure mechanisms that may involve free fall, sliding or rotation of rock blocks. Therefore, discontinuities are critical for identification of potential failure mechanism in fractured rock masses. Important characteristics of discontinuities influencing the strength are orientation, spacing, size & shape of block, roughness, aperture, its in-fillings, wall strength, wall coating, and seepage through them.

A rock mass may display one or more modes of failure depending on following factors:

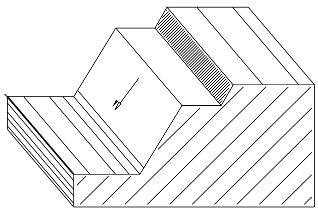
- Presence or absence of discontinuities
- Orientation of discontinuities in relation to that of the natural or excavated face
- Discontinuity spacing in one and three dimension
- Shear strength of discontinuity walls
- Persistence of discontinuities

2.3.1.2 Modes of Rock Slope Failure

Various types of slope failures occurring in cuttings are:

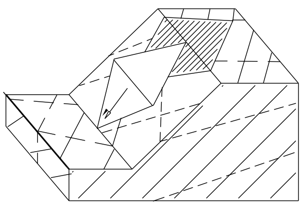
i) **PLANE FAILURE:** It occurs when a geological discontinuity such as a bedding plane, strikes parallel to the slope face and dips in to the excavation at an angle greater than the angle of friction. This is the one of the simplest modes of failure. For plane failure to occur in slopes there must be lateral release surfaces that will allow a block of finite size to slide out of the face. It occurs rarely in rock slopes.

PLANE FAILURE



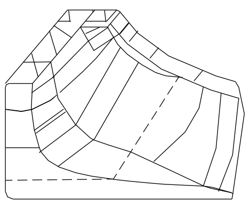
ii) **WEDGE FAILURE:** When two discontinuities strike obliquely across the slope face, the wedge of rock resting on these discontinuities will slide down the line of intersection, provided that the inclination of this line is significantly greater than angle friction. This is most dangerous mode of failure since no release surfaces are required. The calculation of factor in this case is more complicated than that for plane failure since the base areas of both failure planes as well as the normal forces on the planes must be calculated.

WEDGE FAILURE



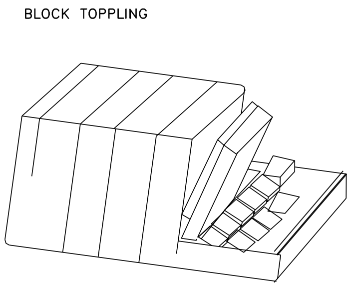
iii) **CIRCULAR FAILURE:** It occurs when material is very weak or rock mass is heavily jointed or broken, the failure will be defined by a single discontinuity surface but will tend to follow a circular failure path. When the pattern of discontinuities is random circular failure modes are likely.

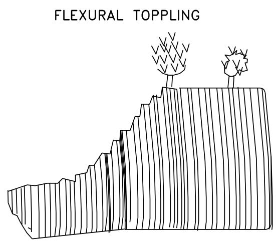
CIRCULAR FAILURE



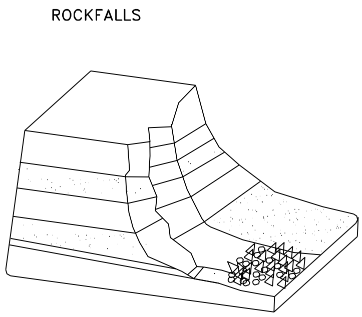
iv) **TOPPLING FAILURE:** It occurs when vector representing the weight of block falls within the base, and inclination of plane is greater than angle of friction. Also, when rock block is tall and slender (height > width), the weight vector can fall out side the base and when this happens the block will topple i.e. it will rotate about its lowest contact edge.

Toppling failure involves either one or a combination of flexural toppling and block toppling. Flexural toppling involves the overturning of rock layers like a series of cantilever beams. Block toppling involves the overturning of fracture-bounded blocks as rigid columns rather than having to fail in flexure.





v) **ROCK FALLS:** Rock falls consist of free falling blocks of different sizes which are detached from a steep rock face. The block movement includes bouncing, rolling, sliding and fragmentation. The problem in design of slopes from view point of rock fall is the prediction of the paths and the trajectories of the unstable blocks which detach from the rock slope so that suitable protection measures are constructed well in advance.



CHAPTER III

ANALYSIS OF LANDSLIDE FAILURES

1.0 The mass movement of rock, debris or earth down a slope is called landslide. Landslide represents only one category of phenomena included under the general heading of mass movements.

The principle points behind landslides are as follows:

(i) Gravity is the principle force involved.

(ii) Movement must be moderately rapid, because creep is too slow to be included as landsliding.

(iii) Movement may include falling, sliding & flowing.

(iv) The plane or zone of movement is not identical with a fault.

(v) Displacement has both vertical and horizontal components of considerable magnitude and movement is downward and outward. It does not normally include subsidence.

(vi) The displaced material should have well-defined boundaries and usually involve only limited portions of hill slide.

2.0 <u>LANDSLIDE FEATURES</u>

A typical diagram showing the features for a complex earth slide / earth flow has been given in figure below:

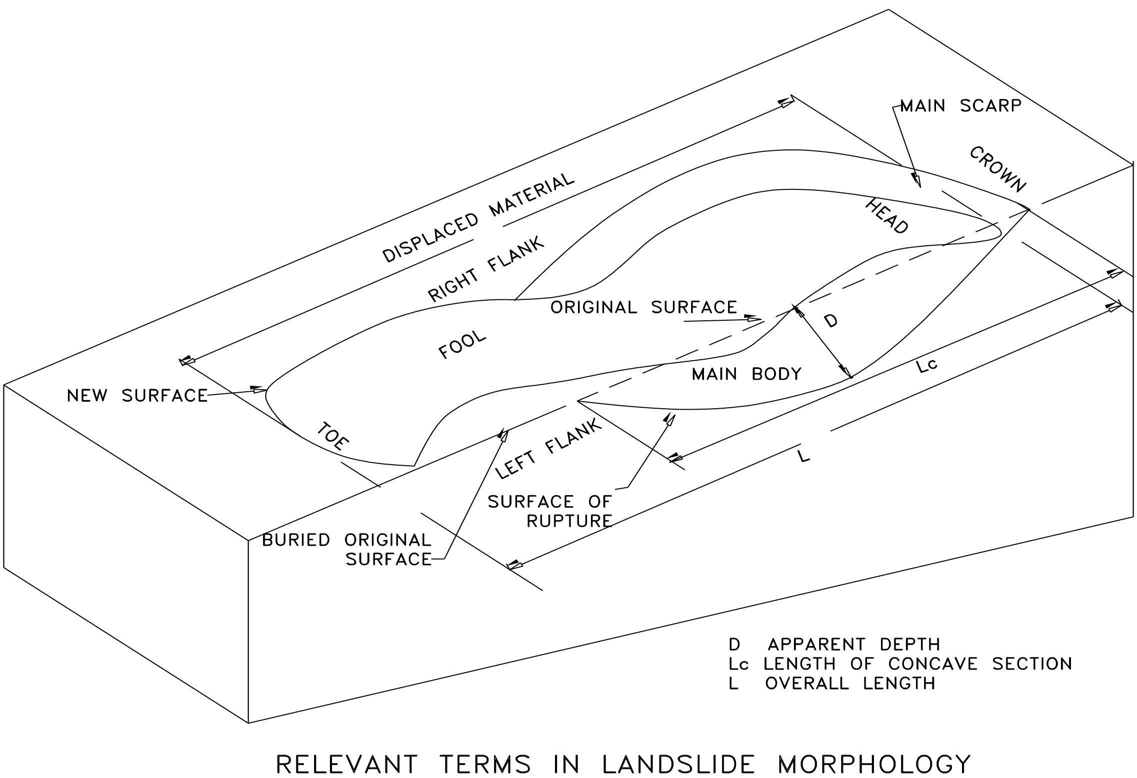
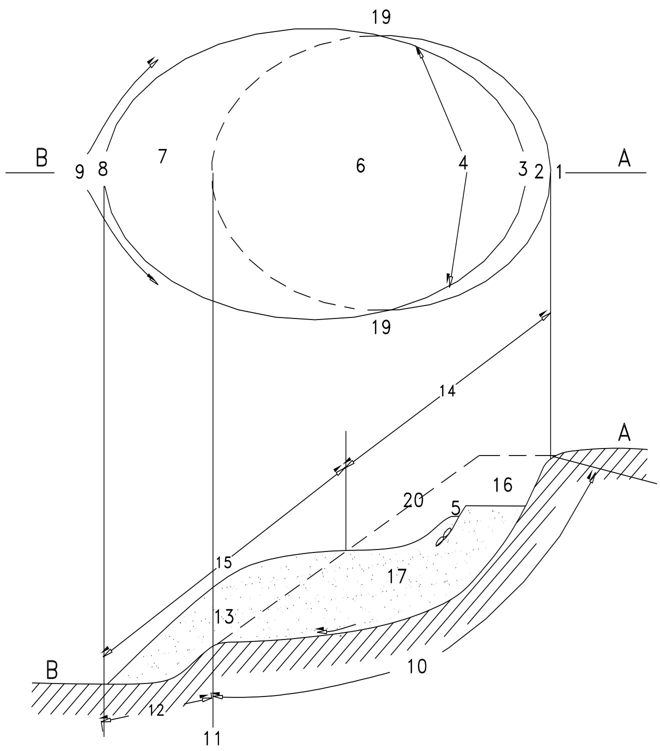


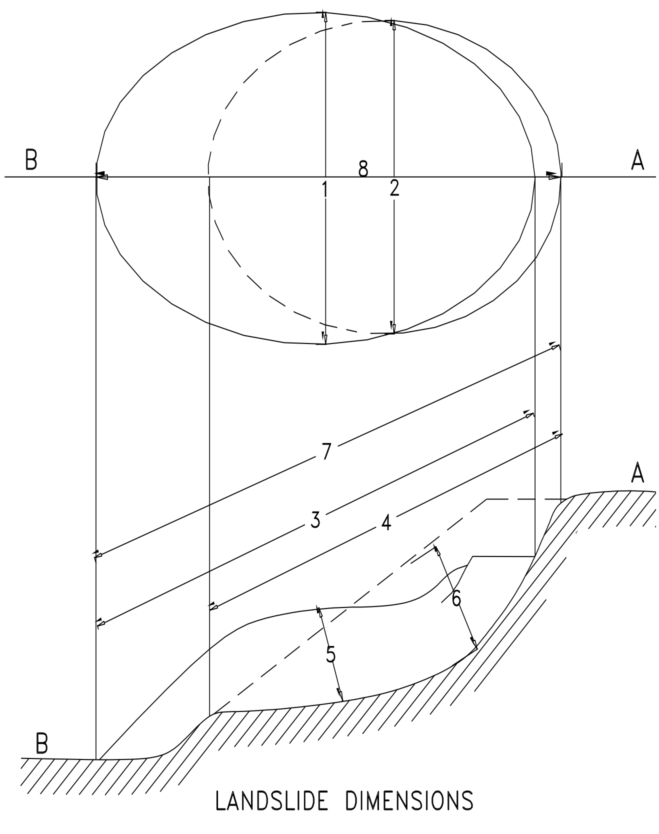
FIG 3.1: LANDSLIDE TERMINOLOGY

TABLE 3.1: DEFINITIONS OF LANDSLIDE FEATURES

S.No.	Name	Definition
1	Crown	Practically un-displaced material adjacent to the highest part of main scarp
2	Main scarp	Steep surface on undisturbed ground at upper edge of landslide caused by movement of displaced material(13 stippled area) away from undisturbed ground, it is visual part of surface of rupture(10)
3	Top	Highest point of contact between disturbed material(13) and the main scarp (2)
4	Head	Upper part of landslide along contact between displaced material and main scarp (2)
5	Minor scarp	Steep surface on displaced material of landslide produced by differential movements with displaced material
6	Main body	Part of displaced material of landslide that overlies surface of rupture between main scarp(2) and toe of surface of rupture (11)
7	Foot	Portion of landslide that has moved beyond toe of surface of rupture(11) and overlies original ground surface (20)
8	Tip	Portion on toe(9) farthest from top (3) of land slide
9	Toe	Lower, usually cured margin of displaced material of a landslide, most distant from the main scarp(2)
10	Surface of rupture	Surface that forms lower boundary of displaced material(13) below original ground surface (20), mechanical idealization of surface of rupture is called slip surface
11	Toe of surface of rupture	Intersection between lower part of surface of rupture(10) of a landslide and original ground surface (20)
12	Surface of separation	Part of original ground surface (20) now overlain by foot (7)of land slide
13	Displaced material	Material displaced from its original position on slope by movement of landslide
14	Zone of depletion	Area of landslide with in which displaced material(13) lies below original ground surface(20)
15	Zone of accumulation	Area of landslide with in which displaced material(13) lies above original ground surface(20)
16	Depletion	Volume bounded by main scarp (2) , depleted mass(17), and original ground surface(20)
17	Depleted mass	Volume of displaced material(13) that overlies surface of rupture(10) but underlies original ground surface(20)
18	Accumulation	Volume of displaced material(13) that lies above original ground surface(20)
19	Flank	Un-displaced material adjacent to the sides of surface of rupture
20	Original ground surface	Surface of slope that exited before land slide took place



LANDSLIDE FEATURES (REFERED TO TABLE 3.1)



LANDSLIDE DIMENSIONS

FIG 3.2 : LANDSLIDE FEATURES

3.0 LANDSLIDE DIMENSIONS

The IAEG (Information Technology Applied to Engineering Geology) Commission on Landslide (1990) utilized the nomenclature described in figure 3.2 & table below to provide definitions of some dimensions of a typical landslide.

TABLE 3.2: DIMENSIONAL FEATURES OF LANDSLIDES

S.No.	Name	Definition
1	Width of displaced mass, Wd	Maximum breath of displaced mass perpendicular to the length Ld
2	Width of surface of rupture, Wr	Maximum width between flanks of landslide perpendicular to the length, Lr
3	Length of displaced mass, Ld	Minimum distance from tip to top
4	Length of surface of rupture, Lr	Minimum distance from toe of surface of rupture to crown
5	Depth of displaced mass, Dd	Maximum depth of displaced mass measured perpendicular to plane containing Wd and Ld
6	Depth of surface of rupture, Dr	Maximum depth of surface of rupture below original ground surface measured perpendicular to the plane containing Wr and Lr
7	Total length, L	Minimum distance from tip of landslide to crown
8	Length of centerline, Lcl	Distance from crown to tip of landslide through points on original ground surface equidistance from lateral margins of surface of rupture and displaced material

4.0 CLASSIFICATION OF LANDSLIDES

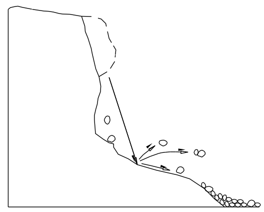
The criteria used in classification of landslides mainly emphasize on type of material and type of slope movement.

TABLE 3.3: CLASSIFICATION OF LANDSLIDES

Type of movement	Type of material		
	Bedrock	**Predominantly coarse**	**Predominantly fine**
Fall	Rock fall	Debris fall	Earth fall
Topple	Rock topple	Debris topple	Earth topple
Slide	Rock slide	Debris slide	Earth slide
Spread	Rock spread	Debris spread	Earth spread
Flow	Rock flow	Debris flow	Earth flow
Complex	Combination of two or more principle types of movement		

4.1 Falls are the separation of soil or, more likely, rock from steep slopes or cliffs. This separation occurs with only minimal shear deformation of the material.

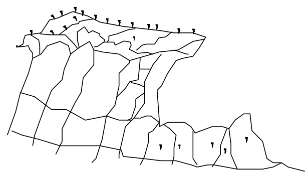
The displaced material may free fall, bound or roll at movement rates which can vary from slow to extremely rapid. In rock fall or debris fall, the moving mass travels mostly through air by free fall, leaping, bounding and rolling with little or no interaction between one moving unit and another. Movements are very rapid.



FALL

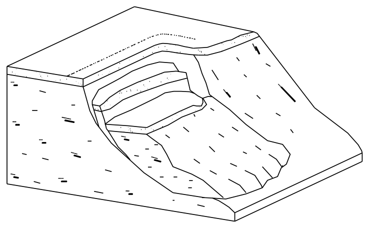
BROKEN LINES INDICATE THE ORGINAL GROUND SURFACES, ARROWS SHOW PORTIONS OF THE TRAJECTORIES OF INDIVIDUAL PARTICLES OF THE DISPLACED MASS.

4.2 Topples are the forward rotation of a near vertical unit (or units) of rock or soil about an axial point at the base of the unit. Movements prior to failure may occur at a slow rate over a long time period. Free fall of the intact unit occurs when gravitation forces exceed the ground strength.



TOPPLE

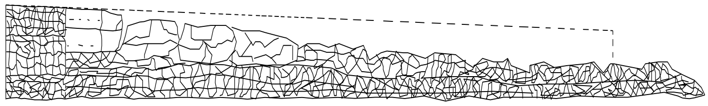
4.3 The term slide implies downslope movement of geologic materials occurring predominantly on either surfaces of rupture or relatively thin zones of intense shear strain. Movement is usually progressive from an area of local failure. The first signs of ground movement are cracks in the original ground surface. The main scarp of the slide may form slowly or rapidly depending on whether the shearing strength of a single unit or the entire mass is exceeded. In slides, significant shearing resistance exists along the shearing surface and movement results from shear failure along one or several surfaces which are either visible or may reasonably be inferred.



SLIDE

BROKEN LINES INDICATE THE ORGINAL GROUND SURFACES.

4.4 Lateral spreads are an extension and runout of the sliding mass. The fractured mass of material may subside into softer underlying material. The rupture surface may or may not be a distinct surface of intense shear. Spreads may result from either liquefaction of cohesionless materials or flow of the softer materials underlying the mass.



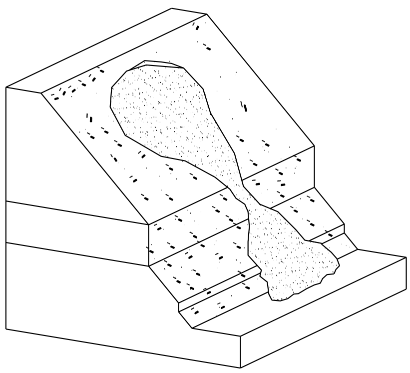
SPREAD

BROKEN LINES INDICATE THE ORGINAL GROUND SURFACES.

4.5 A flow is a viscous movement in which surfaces of shear are short-lived, closely spaced and usually not preserved. The lower boundary of the displaced mass may be a surface along which appreciable differential movement has taken place or a thick zone of distributed shear.

In flows, little or no shear resistance exists along the slip surface and the mass takes on the physical appearance of a viscous material. Generally speaking, slip surfaces are not visible since they are short-lived.

Earthflows in fine-grained soils usually graduate into mudflows at higher moisture content. Generally, the term "debris now " relates to material that contain a relatively high percentage of coarse fragments, whereas the term 'mudflow' is reserved for material with at least 50 per cent sand, silt and clay-size particles. Usually, flow is generated by an unusually heavy rainfall or a cloud-burst or by a sudden thaw of frozen soil.



FLOW

4.6 Very often a slide may include several types of movement within its various parts or at different stages in its development. These are called "complex landslides" . Strictly speaking, most of the actual slides are complex, but at the same time, many of them happen to illustrate vividly only one of the dominant and characteristic types of movement, and as such, they are susceptible to being fitted into the classification system without much difficulty.

5.0 LANDSLIDE INVESTIGATION

Investigation and study of landslides broadly consists of field and laboratory investigations. The objective of these studies is to collect data for the evaluation of the stability of the slope, determine the conditions under which failure may occur and base the remedial measures on a rational footing.

5.1 Field Investigations : Field investigations may be divided into three stages.

a) Mapping of the area
b) Geological investigations
c) Geotechnical investigation

5.1.1 Topographical mapping of the area : The slide area should be mapped in detail. Field maps should be prepared giving the plan of the affected area and typical cross-sections, which can be used for stability analysis. If possible, the topography may be determined by aerial surveys (photogrammetry) which provides an overall view of the site features. General observations should be made concerning the condition of the slope, covering such aspects as the extent and nature of vegetation cover, surface run off characteristics, presence of springs etc. Erosion of the toe and tension cracks in the crown area maybe observed in detail. Any signs or evidences for locating surfaces of failure should be carefully taken note of. Data concerning rainfall and intensity should be obtained as a part of field investigation.

5.1.2 Geological investigations: Geological map of the area, if available, should be studied carefully. Plan of the landslide area must be prepared incorporating geological data. Structural geological features such as bedding planes, joint planes, faults, folds, shear zones etc. should be studied in the field in detail and plotted on the geological map. The influence of these structural geological features on the stability of the affected slope can be evaluated with the help of stereonets, etc.

The rock types in the slide area should be identified and their qualities should be assessed. The minerals in the rocks and their alteration products should be taken into consideration. The investigation must carefully observe for the presence of any soft pockets or beds. In some instances, geophysical studies may help in detecting such layers or pockets.

On the plan of the area already prepared or on a separate map, the geo-morphological features such as elevated and depressed zones, break in slope, erosional and depositional zones etc. should be marked.

5.1.3 Geotechnical investigation : Geotechnical investigations shall be carried out with the objective of determining the nature and strength characteristics of the material comprising the slope. If the slope is predominantly made up of soil or a mixture of soil and rock, disturbed and undisturbed samples should be collected at a few locations covering the affected area. Disturbed samples may be made use of for determining the index properties, grain size analysis etc. Undisturbed samples may be collected from open pits or from boreholes, using appropriate type of sampling tubes. In debris covered slopes, as is very often the case in landslides affected areas of Himalayas, undisturbed samples of good quality can be collected only from open pits. Good quality undisturbed samples are a basic requirement for reliable evaluation of shear strength parameters.

The depth and seasonal fluctuations of water table also form an important component of data required for landslide investigations. This information may be obtained from local enquiries, observations in wells that maybe present or by noting the presence of springs, etc. Sometimes, it may be desirable to make a borehole and install a piezometer, to observe the water level over a cycle of seasons.

5.2 **Laboratory Investigations** : Some of the basic tests that need to be carried out on the soil and rock samples collected from the slide area are as follows-

a) Determination of index properties in case of soil samples.
b) Determination of shear characteristics of slope material by appropriate type of shear tests. If the material is by and large fine-grained, triaxial shear test maybe suitable. If sample contains relatively high content of gravel or rock fragments, direct shear test could be conducted more easily on such samples. The size of the samples used for testing would depend upon its granular composition. The shear strength parameters should be determined using a test procedure compatible with the method of analysis.
c) Rock sample should be examined to find out the nature of rock, extent of weathering, presence of any weak inter layer etc. If suitable samples are obtained, strength of rock samples may also be determined.

5.3 The above data are used for stability analysis and formulation of corrective measures.

6.0 ROCK FALLS

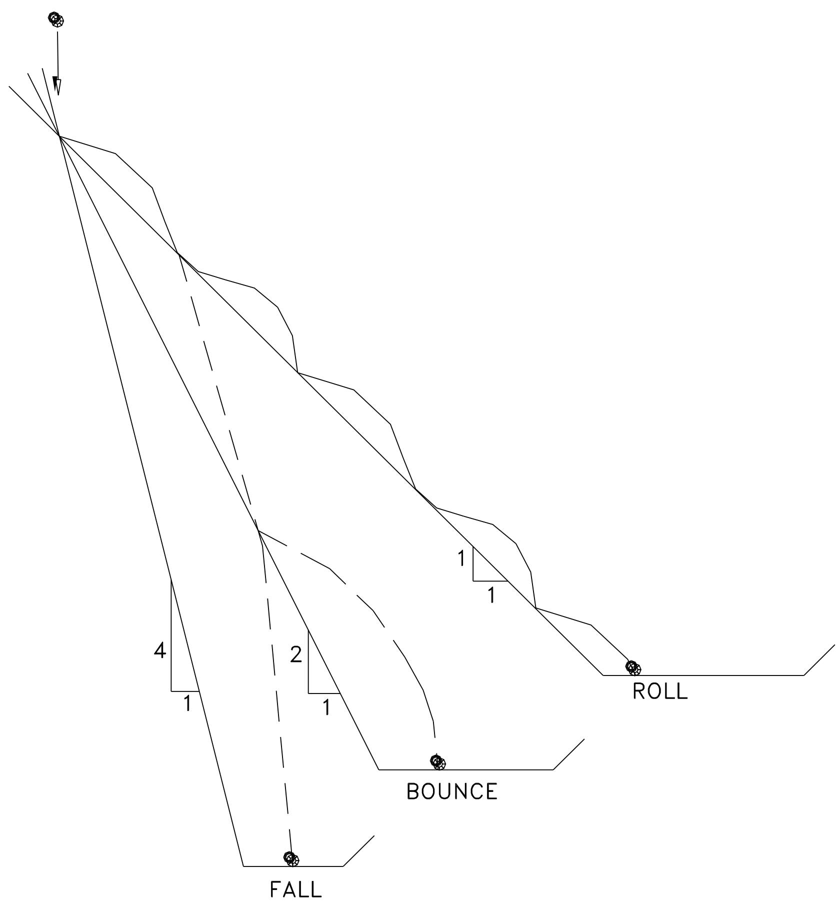
6.1 Rockfalls are generally initiated by some event that causes a change in the forces acting on a rock. The events that bring about rockfalls may include:

- pore pressure increases due to rainfall infiltration
- erosion of surrounding material during heavy rain storms
- freeze-thaw processes in cold climates
- chemical degradation or weathering of the rock
- root growth or leverage by roots moving in high winds
- vibrations due to earthquakes or construction activities
- Change in environmental features causing triggering of destabilizing forces

6.1.1 In an active construction environment, the potential for rockfall will probably be one or two orders of magnitude higher than in case of climatic and biological initiating events described above.

6.2 Once rockfall has been initiated, the most important factors controlling its trajectory are the geometry of the slope and type of slope surface. Other factors of lesser importance are block size and shape, frictional characteristics of the rock surfaces and whether or not rock the block breaks on impact.

6.2.1 Slopes that act as 'ski jumps' are most dangerous as they impart a high horizontal velocity to the falling rock, causing it to bounce a long way out from the toe of the slope. Figure 4.1 shows the nature of rock trajectories for different slope angle. As shown in the figure, loose rocks and boulders from the face of the slope can fall, bounce or roll down the slopes.

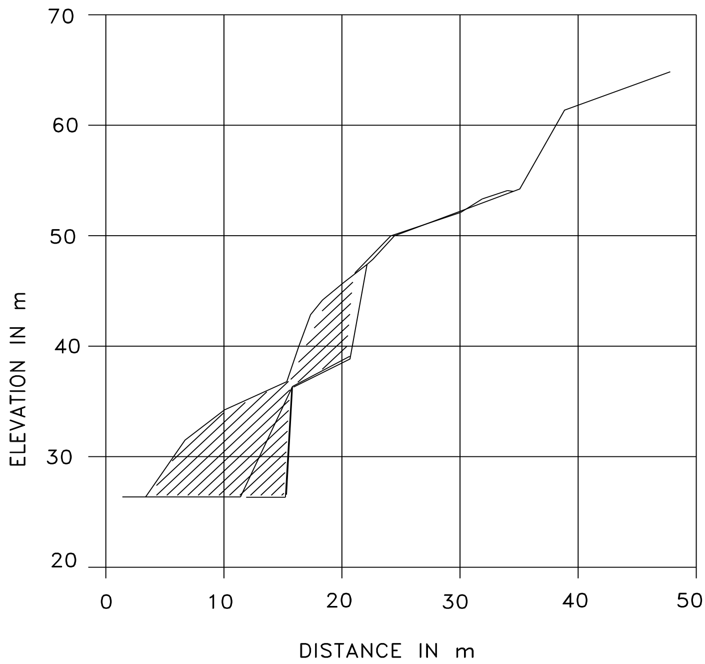


PATH OF ROCK TRAJECTORY FOR VARIOUS SLOPE ANGLES

Fig 4.1

6.2.2 The retarding capacity of the surface material is expressed mathematically by a term called the *coefficient of restitution*. The value of this coefficient depends upon the nature of the materials that form the impact surface. Clean surfaces of hard rock have high coefficients of restitution and do not retard the movement of the falling rock to any significant degree while talus material, gravel and completely decomposed granite have low coefficients of restitution and absorb a considerable amount of energy of the falling rock and many times stop it completely. This is why gravel layers are placed on catch benches in order to prevent further bouncing of falling rocks.

6.2.3 Results of rockfall analysis based on simulation studies is reproduced in figure 4.2 below.

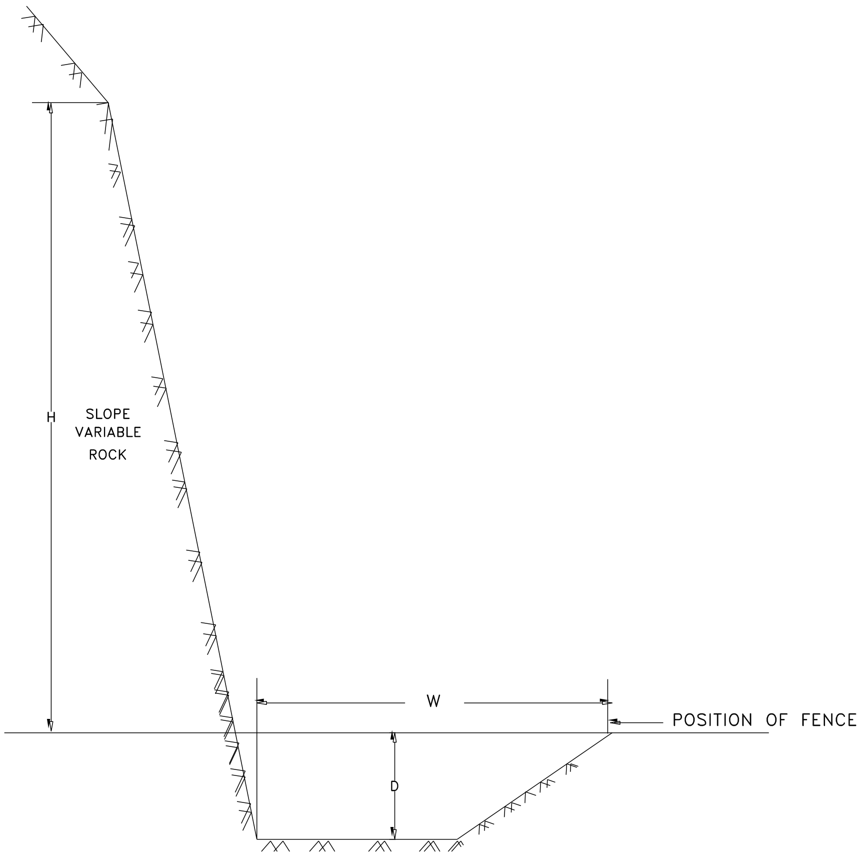


TRAJECTORIES FOR 1000 BOULDERS WEIGHING BETWEEN 200 AND 20000kg,

Fig 4.2

6.3 Rockfalls can attain a very high velocity, which may be as high as 200 kmph, if large rock masses is separated high from a mountain. As a result, falling masses possess a high kinetic energy and, unless steps are taken to dissipate the energy, which has been acquired by the boulder, considerable damage can be caused.

6.3.1 A ditch at the foot of the slope as shown in figure 4.3 will contain much of the energy of the fall and a chain link fence on the shoulder of the trench will prevent the rock from bouncing on to the track. Bedrock should not remain exposed in the bottom of ditches, but should be covered with small broken rock, gravel or sand for dissipating energy of falling rocks and to keep them from bouncing or shattering.



DESIGN CRITERIA FOR DITCHES

Fig 4.3

6.4 Details of ditches and other methods of rockfall prevention have been discussed in subsequent chapters.

CHAPTER IV

DESIGN OF CUTTINGS

1.0 Design of cuttings for railway track is the most important control point for a safe and reliable train operation. It involves the design of following components:

a) slopes
b) formation with or without blanket and geosynthetics
c) erosion and slope protection works
d) side drains
e) boundary drains

It is assumed that the general project alignment has already been selected, field exploration and soil sampling have been performed and laboratory testing is completed prior to commencing the final design. The cuttings more than 25 m depth are generally avoided to the extent possible.

1.1 It will be useful to note the dip and strike of planes of discontinuities. Data needed for estimation of RMR and RQD should also be collected.

1.2 Any seepage occurring in the area needs to be mapped and analysed for possible percolation/saturation. Seepage points mapping may help in identifying the source of ingress of water.

2.0 COMPONENTS OF CUT SECTIONS

The construction of cuttings is done from top to bottom, however design proceeds from bottom to top. A cut section has following components-

- bottom width which includes the formation, side drains, catch ditches, trolley refuges, retaining walls etc.
- side slopes which will include stable slope design, benching or berms, contour drains, pitching and erosion protection works etc.
- top which will incorporate catch water drains and slope stabilization for the terrain.

A typical cut section detailing the considerations for bottom width is produced below.

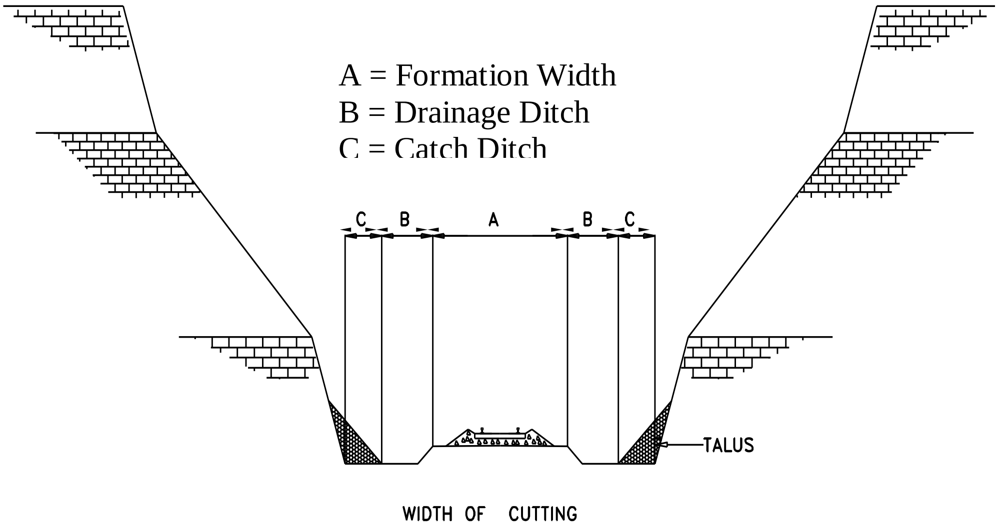


FIG5.1 : COMPONENTS OF BASE WIDTH OF CUTTINGS

3.0 DESIGN OF BOTTOM WIDTH

Design features and their applicability over the cut section are tabulated in the table below.

TABLE 5.1: FACTORS AFFECTING BOTTOM WIDTH OF CUTTINGS

Segment	Purpose	Where provided	Width & Profile
A Formation Width	To provide base for supporting ballast, sleepers and rail	Throughout cut	Standard width as per Rly. Bd. 86/W2/Misc./0/26 dt 25.3.91
B Drainage Ditch	To carry run-off from water shed served, seepage entering cut, and for track	Throughout cut	Standard width profile that may have to be steeper than track profile in long level cuts.
C Catch Ditch (Optional)	To contain material which may fall from faces of cut, and for service and maintenance access	Vulnerable location having broken or rapidly weakening rocks cuts	Of variable width depending on slope and height of cut face, size and rate of fall fragments and desirable frequency of ditch cleaning. Primary consideration in setting width is to position the toe of slope at a point that will not allow falling fragments to bounce into track area. Working

			width required by ditch cleaning machines is important.

3.1 Formation width & design

The design of formation width and other details are covered in RDSO's booklet titled, "Guidelines for Earthwork on Construction Projects-2002". As per the orders of Railway Board, the minimum formation width inside cuttings should be 6.25 m on single line BG routes and 11.55 m for double line.

3.1.1 In deep and long cuttings, it will be more useful to increase the formation width by 4 m (2 m on either side of side drain) for safe running of trains and proper maintenance.

3.2 Side drains

Side drain width is generally standard 1.2 m wide on top, 0.6 m at bottom. The depth is min 0.3 m, with deeper drains as per longitudinal slope depending upon length of cutting. Sub-surface longitudinal drains may be required where blanket layer has been provided.

3.3 Catch ditches

Ditches are important tools for containing the detached rock mass so as to prevent them from reaching the track and creating obstruction over track. The slope, depth, width & storage volume are important factors in design of ditch. The choice of ditch geometry should also take into consideration the angle of slope that influences the behavior of falling rocks. The criteria that involve the height and angle of slope, depth of ditch and width of fallout area are given in table below. If the space is less than required, limited protection against small rolling rocks can still be provided inexpensively by excavating a catchment area or by installing a low barrier formed of gabions at the shoulder of the track.

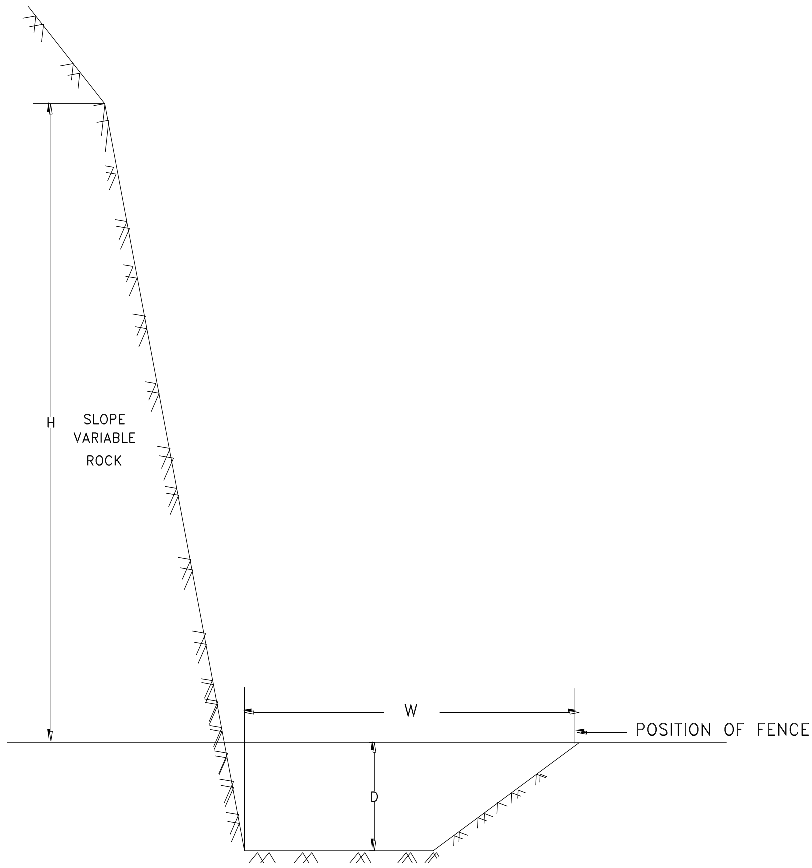


FIG. 5.2: A TYPICAL CATCH DITCH

TABLE 5.2: DESIGN CRITERIA FOR CATCH DITCHES (RITCHIE, 1963)

Slope	Height(m)	Fallout area width(m)	Recommended ditch depth(m)
Near vertical	4 to 10	3.7	1.0
	10 to 20	4.6	1.2
	> 20	6.1	1.2
0.25 or 0.3:1	5 to 10	3.7	1.0
	10 to 20	4.6	1.2
	20 to 30	6.1	1.8*
	> 30	7.6	1.8*
0.5:1	5 to 10	3.7	1.2
	10 to 20	4.6	1.8*
	20 to 30	6.1	1.8*

	> 30	7.6	2.7
0.75:1	0 to 10 10 to 20 > 20	3.7 4.6 4.6	1.0 1.2 1.8*
1:1	0 to 10 10 to 20 > 20	3.7 3.7 4.6	1.0 1.5* 1.8*

* can be reduced to 1.2 m if a catch fence is used where there is space constraint.

3.3.1 In rockfall zones where benches cannot be provided at intermediate levels due to various constraints, provision of catch ditches is important to prevent rock fragments from reaching the track. In such situations, ditches should be designed and provided with ample width at the base of slope to collect rockfall/slip material. The falling rocks striking a slope flatter than vertical will receive a horizontal component of force tending to throw them toward the track area. For this reason, in rockfall zones the slope should be kept as nearly vertical as possible consistent with overall stability.

3.2.2 In approaches of major bridges and tunnels where rockfalls could prove to be catastrophic, provision of such catch ditches should be considered on priority.

3.3.3 Unless mature cuts in similar rock can be observed, it is difficult to predict at the design stage the manner in which rocks will fall and how fast they will accumulate at the base of the slope. Hence, ditches should be designed with ample width to collect rockfall material, keep it out of the track area and permit economical removal of debris. The cost of enlarging catch ditches later in rock cuts is prohibitive.

4.0 DESIGN OF SIDE SLOPES

Slope stability analysis is performed to aid in selecting the steepest safe slope. Cross-sections should then be drawn transverse to the proposed track alignment to determine if safe cuts can be made within the right-of-way lines or if additional right-of-way or slope reinforcement will be required for the project. Soils and rock materials may necessitate that the slope be cut at varying slopes.

Moreover, as excavation progresses, detailed observations are to be made to determine whether the excavation has disclosed any new aspect of geologic structures. Appropriate instruments to detect zones of movement and to verify ground water conditions may also be needed, and modifications to design may prove necessary. Even so, the possibility of a slide must be anticipated and its consequences considered.

4.1 Reliability of slope stability analysis

Adequate analytical procedures have been developed in soil and rock mechanics for calculating the factor of safety of slopes. However, results of such analysis may not have any relation to reality in case of rocks unless all the following conditions are satisfied:

i) geologic profile is reliably known
ii) relevant physical properties of materials along potential surface of sliding have been established;
iii) locations of features such as faults, shear zones, and joints have been determined;
iv) shearing resistance along such discontinuities is known, including the effects of any in-filling or coating materials and including knowledge of prior movements that might have reduced the strength to residual values; and
v) values have been ascertained for pore pressures that may be expected on the surface of sliding under normal and unfavorable conditions.

As these requirements are, apart from being costly affair, difficult to be satisfied and involve a considerable element of uncertainty, design based on stability analysis is justifiable only for occasional critical cuts.

Following tables summarize various methods of slope stability analysis - conventional as well as modern numerical methods along-with their advantages and limitations.

TABLE 5. 3 : CONVENTIONAL METHODS OF SLOPE STABILITY ANALYSIS

Sl. No.	Analysis method	Critical input parameters	Advantages	Limitations
1.	Stereographic and kinematics	Critical slope and discontinuity geometry, representative shear strength characteristics.	Relatively simple to use and give an initial indication of failure potential. Some methods allow identification and analysis of critical key-blocks. Links are possible with other analysis methods. Can be combined with statistical techniques to indicate probability of failure and associated volumes.	Only really suitable for preliminary design or design of non-critical slopes. Need to determine critical discontinuities that requires engineering judgment. Must be used with representative discontinuity joint shear strength data. Primarily evaluates critical orientation, neglecting other important joint properties.
2.	Limit equilibrium	Representative geometry and material characteristics, soil or rock mass shear strength parameters (cohesion and friction) discontinuity shear strength characteristics groundwater conditions reinforcement characteristics and external support data	Wide variety of software available for different failure modes (planar, wedge, toppling, etc). Mostly deterministic but increased use of probabilistic analysis. Can analyze factor of safety sensitivity to changes in slope geometry and material behaviour. Capable of modelling 2d and 3D slopes with multiple materials, reinforcement and groundwater profiles.	Factor of safety calculations give no indication of instability mechanisms. Numerous techniques available all with varying assumptions. Strains and intact failure not allowed for. Do not consider in situ stress state. Probabilistic analysis requires well defined input data to allow meaningful evaluation. Simple probabilistic analysis may not allow for sample /data covariance.
3.	Rock fall Simulation	Representative slope geometry, rock block sizes and shapes, coefficient of restitution.	Practical tool for siting structures. Can utilize probabilistic analysis. 2D and 3D codes available	Limited experience in use relative to empirical design charts.

TABLE 5. 4 : NUMERICAL METHODS OF SLOPE STABLITY ANALYSIS

Sl. No.	Analysis method	Critical input parameters	Advantages	Limitations
4.	Continuum modelling (eg finite element, finite difference)	Representative slope geometry: constitutive criteria (eg. Elastic, elasto-plastic creep etc.) groundwater characteristics, shear strength of surfaces , in situ stress state.	Allows for material deformation and failure. Can model complex behaviour and mechanisms. Capability of 3-D modelling. Can model effects of groundwater and pore pressures. Able to assess effects of parameter variations on instability. Recent advances in computing hardware allow complex models to be solved on PC,s with reasonable run times. Can incorporate creep deformation. Can incorporate dynamic analysis.	Users must be well trained, experienced and observe good modelling practice. Need to be aware of model/software limitations (eg boundary effects, mesh aspects ratios, symmetry hardware memory restrictions). Availability of input data generally poor. Required input parameters not routinely measured. Inability to model effects of highly jointed rock. Can be difficult to perform sensitivity analysis due to run time constraints.
5.	Discontinuum modelling (e.g. distinct element, discrete element)	Representative slope and discontinuity geometry, intact constitutive criteria. Discontinuity stiffness and shear strength, groundwater characteristics, in situ stress state.	Allows for block deformation and movement of blocks relative to each other. Can model complex behaviour and mechanisms (combined mateial and discontinuity behaviour coupled with hydro-mechanical and dynamic analysis.) Able to assess effects of parameter variations on instability.	As above, experience user required to observe good modelling practice. General limitations similar to those listed above. Need to be aware of scale effects. Need to simulate representative discontinuity geometry (spacing, persistence, etc). Limited data on joint properties available.
6.	Hybrid/Coupled modelling	Combination of input parameters listed above for stand alone models	Coupled finite – element/distinct element models able to simulate intact fracture propagation and fragmentation of jointed an bedded media.	Complex problems require high memory capacity. Comparatively little practical experience in use. Requires ongoing calibration and constraints.

4.2 Cuttings in rocks

4.2.1 Major rock cuts require detailed subsurface investigation to know the type & condition of rock strata before taking up the excavation. As and when excavation progresses, additional geological information helps in deciding rock slope, at various levels, by carrying out tests like compressive strength, petrographic examination of samples, soundness tests etc.

The blasting in rock strata plays a very significant role in slope stability. Uncontrolled blasting often results in shattering of rock mass, by means of opening of joints, developments of tension cracks, rough uneven contours, overbreaks, overhangs etc. The results of blast shock wave, along various discontinuities can lead to loosening of the rock.

4.2.2 Rough guide for adopting the slopes or cuts in rock is given in table below. In adopting this table, caution must be exercised and such factors as the influence of dip in relation to the inclination of the slope face, the nature of joints etc. must be kept specially in mind. However, it would lead to safer and economical rock slopes if proper design methods are adopted to evaluate the stability of rock slopes as well.

TABLE 5.5 : TYPICAL DESIGN SLOPES FOR ROCK CUTS

Sl. No.	**Rock Type**	**Range of permissible slope (H : V)**
A.	**Sedimentary Rocks**	
1.	Massive sand stones and lime stones	0.25 : 1 to 0.50 : 1
2.	Jointed/Inter bedded/Layered sand stones, lime stone & shales	0.50 : 1 to 0.75 : 1
3.	Massive clay stone and silt stone	0.75 : 1 to 1 : 1
B.	**Igneous Rocks**	
1.	Massive Granites & Basalts	0.25 : 1
2.	Jointed Granite, Jointed Basalt	0.50 :1
C.	**Metamorphic Rocks**	
1.	Gneiss, Schist and Marble	0.25 : 1 to 0.50 : 1
2.	Slate	0.50 : 1 to 0.75 :1
D.	**Weathered Rocks (All types)**	1:1

4.2.3 Design Information

- Factors that should be evaluated when designing the rock cuts are 3 dimensional competence of the rock and overburden, apart from depth and length of the cut.

- The first steps in design are the preparation of profiles and cross sections on which are plotted data obtained during the site investigation, test borings and laboratory testing, interpreted with the aid of geological maps, groundwater surveys and aerial photographs. The knowledge of the behavior of similar rock in comparable cuts can prove to be valuable design information.

- In layered formations, where dip or strike of the bedding planes is not normal to the center of the cut, it may be desirable to evaluate sections on the dip of the bedding planes to aid in examining the stability of the cut slope.

- A uniform slope in one segment of rock is not necessarily appropriate throughout the length of a cut if the condition of the rock (i.e., strike and dip of bedding planes, fracturing, etc.) changes.

4.2.4 Stability of Side Slopes

- Safe slopes are governed by the characteristics of the rock in the slope. Slope angles should be chosen independently even in the same cut for sound rock, weathered or shattered rock, and overburden.

- For each rock material, the slope is governed to a major degree by bedding planes, joints which are usually perpendicular to the bedding, fracture patterns and faulting, all of which tend to make the rock perform as a number of segments rather than as a mass. The influence of each of these characteristics should be carefully assessed in analyzing slope stability. It should be noted that the slope of such discontinuities, as entered on cross-sections, and profiles show their true angle of interception with the cut slope, which should be considered in design.

- Stability of rock slopes can be analyzed using 3-D slope stability analysis with use of a *stereo net* software program, or by the method of slices (when appropriate) as used with soil slopes, but it should be realized that the surface of sliding will follow rock joints and defects where possible. Values of shear strength (friction angle and cohesion) are chosen accordingly. Cohesion is usually neglected, as its value along joints in rock may be small. Major slopes can be designed only by sufficiently experienced people or through consultants.

- Rock falls and slides commonly occur during or soon after heavy rains, which is an indicator of the major importance of seepage pressures on slope stability. Water has the dual effect of increasing shear stresses in the slope by its weight and hydrostatic pressure, and at the same time decreasing the shear strength of rock materials by weathering, freezing and expansion. Hence, it is important to keep water out of the slope if possible.

- In most rock masses, the ground water table cannot be lowered economically. However, intercepting surface ditches at the top of the slope or horizontal relief drains in the face or at the toe of the slope may have benefits in certain cases.

4.3 Cuttings in Soils

4.3.1 For every soil type, it is necessary to maintain a safe and stable cut section. Berms, drainage, erosion control, filter layers, vegetation and proper selection of the finished cut slope angle should be used as a means of achieving this end.

4.3.2 Common method of calculating factor of safety for earth slopes assuming circular failure surfaces is modified Bishop's method, details of which can be seen in "Guidelines for Earthwork in Railway Projects, July 2003" issued by RDSO.

4.3.3 Considerations such as the proposed slope angle, drainage conditions, and moisture conditions and strength of the soils encountered in cuttings are the most significant factors that influence the stability of earth slopes. All sloping soils have a tendency to move under the influence of gravity. Slope stability evaluations should generally be carried out to select the cross section for cuttings. Observations of nearby cutting in similar soils and natural slopes in the project local can aid in slope design.

4.3.4 Cuts in Cohesionless Soils (Sands and Gravels)

Sands and gravels that are located above the ground water level may generally be provided a slope 1.5(H):1(V) or flatter. Steeper slopes may be able to be excavated and stand for short periods of time, but will eventually try to assume a flatter slope. Finished slopes in sand-gravel materials that are exposed to groundwater flow or seepage from the slope face will routinely have to be cut flatter than would be required for the same cohesionless soil cut in a non-saturated state. In areas of loose saturated cohesionless soils, special provisions may be required to avoid liquefaction. The stability of slopes in sand is generally improved as the density of the cohesionless soil increases.

4.3.5 Cuts in Cohesive Soils (Silts and Clays)

Cuts in cohesive soils need to be designed with caution. Cuts in cohesive soils should invariably be designed using slope stability analysis. Local long-term experience may prove to be an indicator of a stable slope for a particular soil profile. A slope of 1.5 (H):1(V) or flatter generally proves stable in cohesive soils. Clay slopes over 3 to 5 m in height should be designed on the basis of laboratory tests and slope stability analysis. In general, the higher the cut section becomes, the flatter the slope will have to be to remain stable. Highly plastic clay soils require flatter slopes than those discussed above. The stability of clay slopes can be increased by the installation of drains and by flattening the cut slope. Cut slopes in areas where it is known that slides are inevitable may be designed to allow for slope movement (failure) without interference to traffic.

4.3.6 Cuts in Non-Uniform Soils

Cuts in soils which are layered or contain seams of varied soil types should be designed on the basis of a slope stability analysis. The seams that contain cohesionless (granular) soils are often water bearing during some part of the year and drainage of these seams should be provided.

4.4 However, routine adoption of these slopes for rocks and soils is likely to lead to many instances of in-stability that could be avoided or reduced in severity by modifications based on the findings of detailed surface inspections and judiciously executed sub-surface exploration before construction. If such structural features as adverse orientations of structural planes of weakness are disclosed, they should be considered in stability calculations together with results of laboratory or field tests to established the shear strength parameters.

4.5 For laterite cuttings, the matter has been dealt with separately in Chapter titled as "Laterite Cuttings-Construction and Maintenance" and the same may be referred accordingly.

4.6 Benching/Berms

Benching is a design feature of cut sections, where a horizontal platform is introduced in the middle (in-between) of side slopes. Benching reduces the driving forces on a potential or existing slide. Slopes constructed with benches or berms are considered preferable to equivalent uniform straight slopes. Benching produces increased stability by dividing the long slope into segments of smaller slopes connected by benches. The width of benches should be adequate to enable the slope segments to act independently. Benches are useful in controlling instability if they are properly designed and provided with paved drains. Slope flattening by benches is illustrated in figure below.

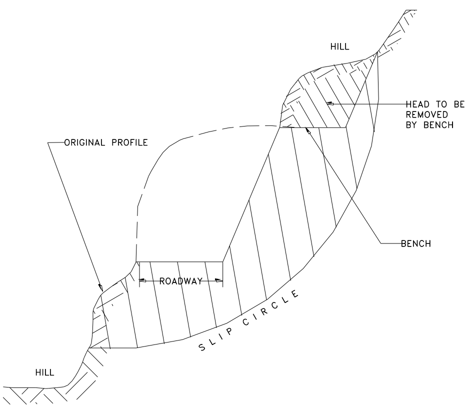


FIG.5.3: REDUCTION OF WEIGHT BY BENCHING

4.6.1 The benefits of benching are multifold, such as:

- Break in monotony of slopes and reduce pressure at the toe of cuts, thereby averting slope failure due to slip circle mechanism
- Provide space for accommodating falling debris and rock
- Minimize rock-falls during and after construction
- Accommodate contour drains
- Break the speed of water along the slope and reduce erosion
- To prevent undermining of hard strata by differential weathering
- Provide access and working space for inspection and maintenance.

Benched slopes are considered an expensive option because of increase land-take and construction time. However, the advantages of a benched slope more than justify the expenses.

4.6.2 General Principles

- Benching may be internal, i.e. within the cut slope or external, i.e. by provision of gabions or retaining walls.

- The geometry of benches is governed by physical and mechanical characteristics. Bench height can be greater in stronger rock, and the bench face can be terminated at the base of weaker horizons and water bearing zones. Bench height, berm width, bench face angle, overall slope angle & form of excavated slope incorporating benches is shown in figure below-

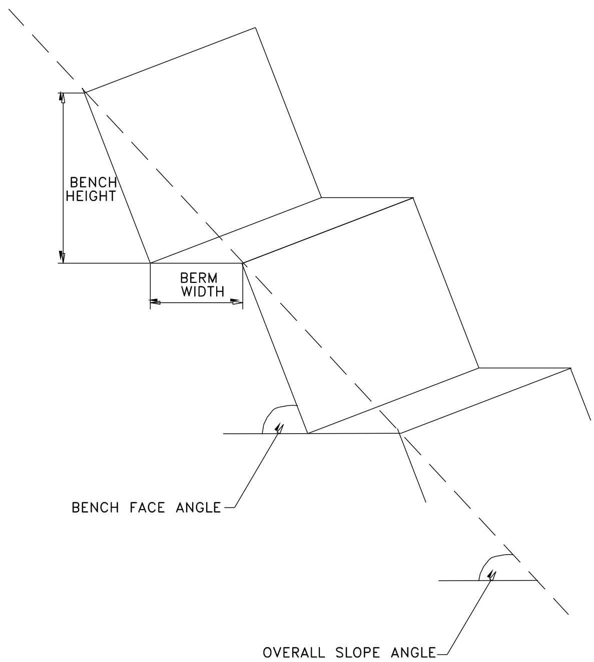


FIG.5.4:BENCH DESIGN PARAMETERS

- Without affecting the overall slope angle, higher benches generally will allow for wider berms, giving better protection, more reliable and easier access for the regular cleaning of debris.

- Design of bench face angle is governed to a large extent by the attitude of unfavorable structures in the slope to prevent excessive rock fall on to the berms. If the bench faces are nearly vertical, high tensile stresses are likely to develop near bench crests causing tension cracks & over hangs to form. Ideally, bench faces should be inclined.

- Berms should be equipped with drainage ditches to intercept surface run-off and water from drain holes and other drainage facilities and divert it off the slope and away from problem areas. Berm surfaces should be graded to assist the collection of water in ditches and also to facilitate general drainage in a direction away from potential areas of instability.

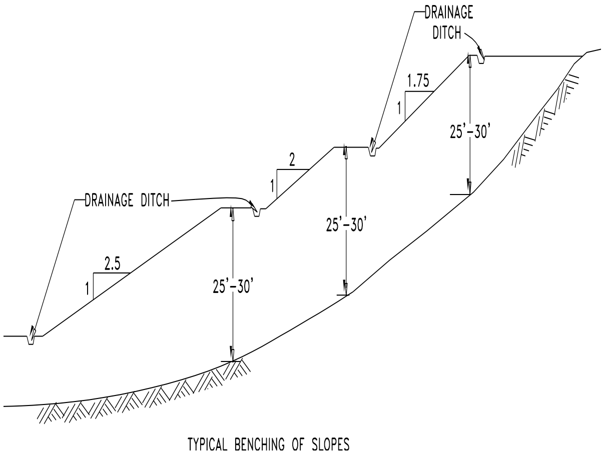


FIG. 5.5: BENCH WITH DRAINS

4.6.3 <u>Berms in Rocks</u>

- Where permanent benches are used to intercept falling rock, access should be provided for periodic removal of debris. The width of such benches may be decided on the basis of mechanical working after weathering of the softer rock has taken place. A minimum width of 5 to 6 m may be desirable.

- In shales and other soft-rock cuts, temporary benches may be designed to contain all debris from a steep slope.

- Benches used to reduce the effects of differential weathering are located at the top of the weaker rock where the stronger rock is set back to form the bench. The width of the bench is governed by the weathering characteristics of the weaker rock and the height and angle of its slope. Provision for access may not be required.

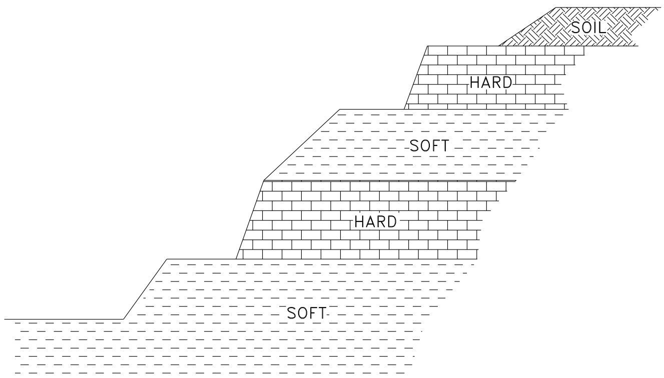


FIG. 5.6: BENCH IN ROCK

- In deep rock cuts, where weaker rock appears in the base of the cut (such as laterites), it may be necessary to introduce benches to relieve the toe pressure. Such benches may serve other purposes, as noted above, to increase safety and reduce maintenance costs.

- Drainage of benches is best accomplished by sloping them to the face of the cut thus moving water off the bench as quickly as possible. Where rock on the surface of the bench presents open joints or fractures, water may be prevented from entering the rock mass by covering the bench with a layer of clay or other impervious material, thus reducing or eliminating deterioration of the rock by ice wedging and erosion.

- Stepped benches may be used on slopes cut in highly weathered rock material to control erosion and to establish vegetation as shown in figure below. It consists of 0.6 to 1.2m high benches with approximately similar berms width and an overall slope angle based on stability analysis. The design objective is that the material weathering from each rise will fill up the step of the bench and finally create a practically uniform overall slope. The steps are constructed horizontally to avoid the longitudinal movement of water, which could cause considerable erosion.

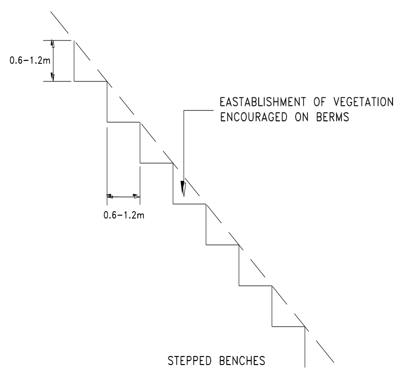


FIG. 5.7 : STEPPED BENCHES

4.6.4 Berms in Soil Slopes

It is advisable to provide berms in soil slopes at every 6 to 7 m height to break monotony of slopes. Width of first berm from formation may be kept as 5 m and that of subsequent berms as 4 m.

4.6.5 Berms at Soil-Rock Interface

In cuttings where soil-strata is in top portion and weathered/jointed rock in bottom portion, it is essential to provide 5 to 6 m berm at soil-rock interface.

4.6.6 Pitching and Erosion Protection

Erosion and slope protection have been dealt in detail in subsequent chapter no V.

5.0 BOUNDARY DRAINAGE

Rainfall, percolation or streams that flow outside the boundaries of the cutting have a grave potential of affecting the behaviour of the cutting in immediate or even distant future. Effective steps need to be taken to avert any eventuality of ingress of any such water that has not been catered for in the design. This job is to be accomplished by provision of boundary or peripheral drains. These drains are also called catch water drains. Design aspects of these drains have been discussed in subsequent chapter.

CHAPTER V

DRAINAGE & EROSION CONTROL

A. DRAINAGE

1.0 Ingress of water is the principal triggering factor for slope instability and landslides. It is the water pressure rather than the quantity of water which matters in the stability of the slope. For effective control of slides, water presence and its flow needs to be taken care of. Improvement in the surface or sub-surface drainage conditions increases the stability of the slope, therefore, it should be given high priority in the design and maintenance of cuttings. Drainage measures, as compared with other possible measures, frequently result in substantial benefits at significantly lower cost. Often large failures cannot be controlled within practical limits by any means other than some form of drainage. Therefore, it is important to study the ground water regime and adopt appropriate surface and subsurface drainage techniques to improve unfavourable conditions.

2.0 During sustained rainstorms of high intensity, general rise of water table following the recharge in catchment areas as well as direct infiltration of water, lead to reduction in effective stress due to increase in pore water pressure and hence reduced shearing resistance. Therefore, relative depth of the water table and the infiltration potential in each morphological unit, which essentially depends on the runoff and permeability of the outcropping formation, must be given due consideration in design.

Rock slope instability is mainly due to discontinuities (joints, bedding planes, fractures and faults). The rain and surface water, seeping out at a number of places from the contacts of boulder beds and rock results in increased joint water pressure during monsoon and considerably reduce the cohesion of the rocks. As a consequence, highly jointed and sheared rocks are detached and slides occur. Prolonged intense rainfall and treacherous unstable geological settings such as presence of permeable and unstable joints, dykes, unfavourable bedding planes etc aggravate the problem.

Leaching of the slopes by seepage may cause changes in soil chemistry, for example a Calcium (Ca) clay converting to Sodium (Na) clay, affecting slope stability. Most natural slopes of residual soil exist in an unsaturated condition and their margin of safety against sliding depends on the capillary tensions which exist in the pore water that enhance the strength of the soil. Infiltration during prolonged rainfall can reduce capillary tension to a point where the slope becomes unstable. The increase in density caused by the increased water content further aids sliding.

3.0 Drainage and removal of water are generally the principle measures used for controlling landslides in a wide range of both soils and rocks. Drainage measures are required to minimise surface water infiltration on the slope and reduction of the pore water pressure rise in the slope resulting from recharge through permeable beds/seams.

Drainage measures taken to control slides are as follows:

- Prevention of seepage/infiltration of water.
- Removal of the obstructions to the flow of natural water courses.
- Provision of an adequate drainage system in the vicinity of the potential failure surface to tackle surface and subsurface water.
- Position the drainage so that it reduces the water pressure in the immediate vicinity of the slope.

Drainage not only reduces the weight of the mass tending to slide but also increases the strength of the slope-forming material. In absence of proper drainage facility, development of excess pore water pressure within the slope leads to slope instability. Surface drainage must be part of any design scheme, however, subsurface drainage will be effective only at those sites where water table is above shear plane.

The effectiveness of any slope drainage scheme is very difficult to gauge. Piezometers installed in the slope before the drainage system is brought into operation can give very valuable information on the reduction of water pressure which is achieved. Knowledge of overall groundwater flow patterns is very important in planning the most effective drainage measures. Drainage, however inefficient, is better than no drainage.

4.0 In presence of water, a stable slope can become unstable as the shear strength is reduced by:

- Additional wetting of soil mass as a result of excessive infiltration by surface runoff or ground water.
- Loss of suction or decrease of strength as a result of saturation.
- Gradual loss of strength due to loss of fines through seepage flow weathering process.

Alternatively, a stable slope can become unstable as the shear stress is increased by:

- Increase in weight due to added water in soil mass as a result of excessive infiltration or inadequate surface drainage during prolonged intense rainfall.
- Seepage force due to creation of hydraulic head or rise in water table
- Steepening the slope by erosion

5.0 Most of the landslides or slope failures are triggered by prolonged intense rainfall. Heavy rainfall usually results in excessive infiltration especially when the soil is very permeable and adequate drainage is not provided and slope vegetation is not complete or ineffective. When the rainfall intensity increases, the entire soil layers are saturated faster.

Groundwater doesn't emerge in the soil from the slope, unless the average saturation ratio exceeds a certain minimum value. When the saturation ratio is lower than this value, the rain water dropped on the soil is retained in it to complement its insufficient water content, irrespective of the amount of rainfall. Whatever be the volume of rain water, the relation between groundwater level and saturation ratio generally makes convex curve as shown below.

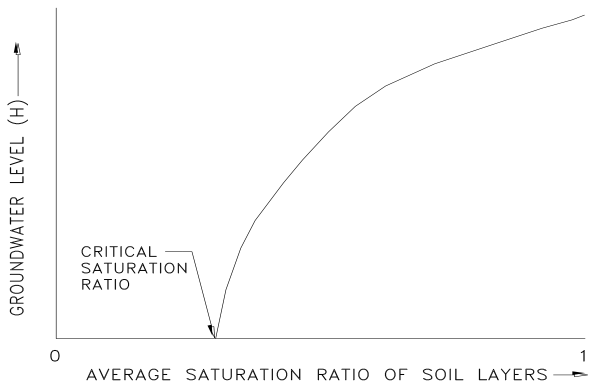


FIG. 9.1: GROUND WATER VS SATURATION RATIO

The curve above shows that the groundwater level rapidly rises after the average saturation ratio over soil layers has passed the critical minimum value for emergence of groundwater and that its rising speed sharply drops after a certain point.

After the start of rainfall, till the time groundwater does not emerge, the safety factor declines slightly. This is the result of the increased soil weight due to the infiltration of rain water into the slope. The groundwater level starts to sharply rise after certain hours after the start of rainfall. This increases the pore water pressure, the safety factor drops to below 1 and the same may result into landslide. After the rainfall has stopped, the groundwater level drops and the safety factor recovers as shown in the figures below-

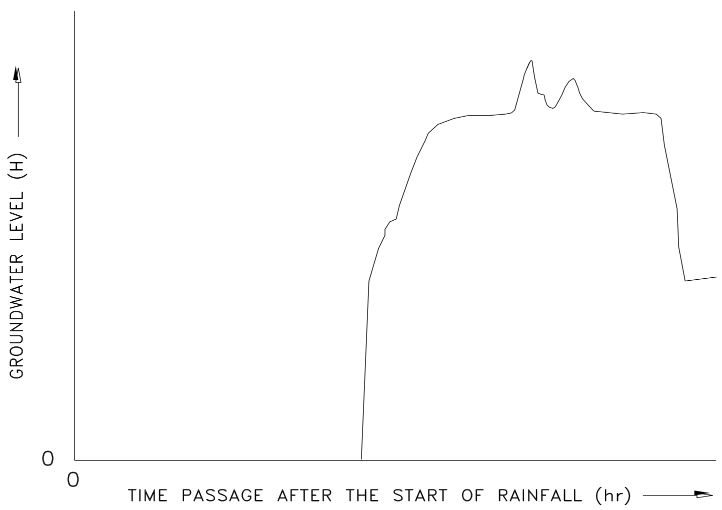


FIG. 9.2: TRANSITION OF GROUNDWATER LEVEL (H)

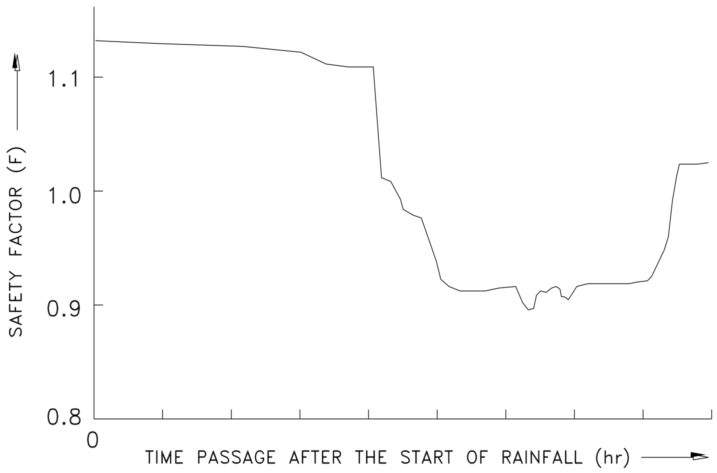


FIG. 9.3: TRANSITION OF SAFETY FACTOR (F)

Thus, the groundwater level largely affects the safety factor and subsequently the occurrence of landslide.

6.0 SURFACE DRAINAGE CONTROL

Adequate surface drainage facilities, particularly if the rocks are relatively soft or susceptible to erosion, can substantially improve the stability of a slope. Areas behind the upper portions of unstable slopes should be thoroughly inspected to determine whether surface water is flowing toward unstable areas.

All precautions should be taken to prevent the surface run-off from entering a potentially unstable area. Provision of surface drains along the outer periphery of the potentially unstable area is particularly desirable. Surface drains serve to intercept the run-off from higher ground, and divert it away from the slide surface. If surface drains are likely to be clogged by debris from above, a drain pipe should be placed to ensure that the water is not trapped inside the area. The surface drains must be provided with impervious paving and have a uniform gradient to prevent deposition of material silt on the bottom of the drain.

Mere provision of drains, culverts and drainage chutes does not help if it is not ensured that the catchment would effectively feed them. A broken or dislocated surface drain may sometime cause damage to the slope more than the lack of any drainage. Surface drains should be located very carefully after the topography of the ground is studied. Poor location of surface drains results in their serving no purpose, since no water would be collected by them. On the other hand, the run-off by-passes the drains and continues to damage the slopes.

In nutshell, the following methods may be used to control surface drainage:

- The upper slope surface, immediately behind the crest, is an area of considerable potential danger since water which is allowed to pond in this area will almost certainly find its way into the slope through open tension cracks and fissures. Grading of this surface and the removal of piles of waste rock or over-burden which could cause damming will enhance run-off of any collected water.
- Ensure drainage of water-filled depressions from which water could seep into unstable zones.
- Reshape the surface of the area wherever necessary to provide controlled flow and surface run-off.
- Above the crest of the slope, seal or plug the tension cracks and other highly permeable area that may appear to provide avenues for excessive water infiltration.
- Provide surface ditches, culverts, surface drains or conduits to divert undesirable surface flows into non-problem areas.
- Minimize removal of vegetative cover and establish vegetative growth.

6.1 Side Drains

Inadequate drainage on hill side causes softening of the sub-grade and renders it too weak to take the load of the moving traffic. Side drains are therefore necessary on hill side. Side drains are provided along the hill side for taking the surface run off to the nearest cross drain. These can be taken up for construction after stabilization of hill slopes to some extent. These drains should be lined as the water flow is likely to erode the bed and sides of the drain. In inhabited areas, the lining of drains could be on aesthetic reasons also.

6.1.1 Keeping in view the convenience of construction, it may be necessary to have uniform section of a drain but the frequency of culverts could be regulated to the catchment area that it has to cater to.

6.1.2 To discharge runoff from hill side drain to valley side, adequate number of culverts can be provided or the side drains may be connected to discharge into natural water course.

6.1.3 For side drains, the parabolic (saucer shape) section is hydraulically the best and most erosion resistant. However, the trapezoidal section is more generally used as it is easier to construct.

6.1.4 Generally, gradient of drains should be sufficient to develop self cleansing velocity to disperse floating debris conveniently. In continuous long stretches with steep grades, the side drains should be stepped to break the velocity.

6.2 Catch Water Drains

Drains provided on hill side away from the track to intercept and divert the flow water before it reaches the side drain are known as catch water drains. They collect run-off from upper reaches of hill slopes and guide it into culverts before it reaches the area immediately behind the crest of the slope. This is the area in which the most dangerous tension cracks are likely to occur. Their location, size, gradient and lining helps in checking potential slides.

6.2.1 A number of rows of inter-connecting lined catch-water drains should be constructed on the slope to collect the surface run off which should in turn be brought to culverts at a lower level to be led through chutes to natural water-courses.

6.2.2 Such catch-water drains should be provided in stable hill slopes outside the periphery of slide/unstable areas so that stability of hill is not further worsened. In such cases, additional intermediate drains may also have to be provided in some cases depending on ground conditions. Figures given below depict catch-water drain arrangement on a stable hill slope and in a slide area.

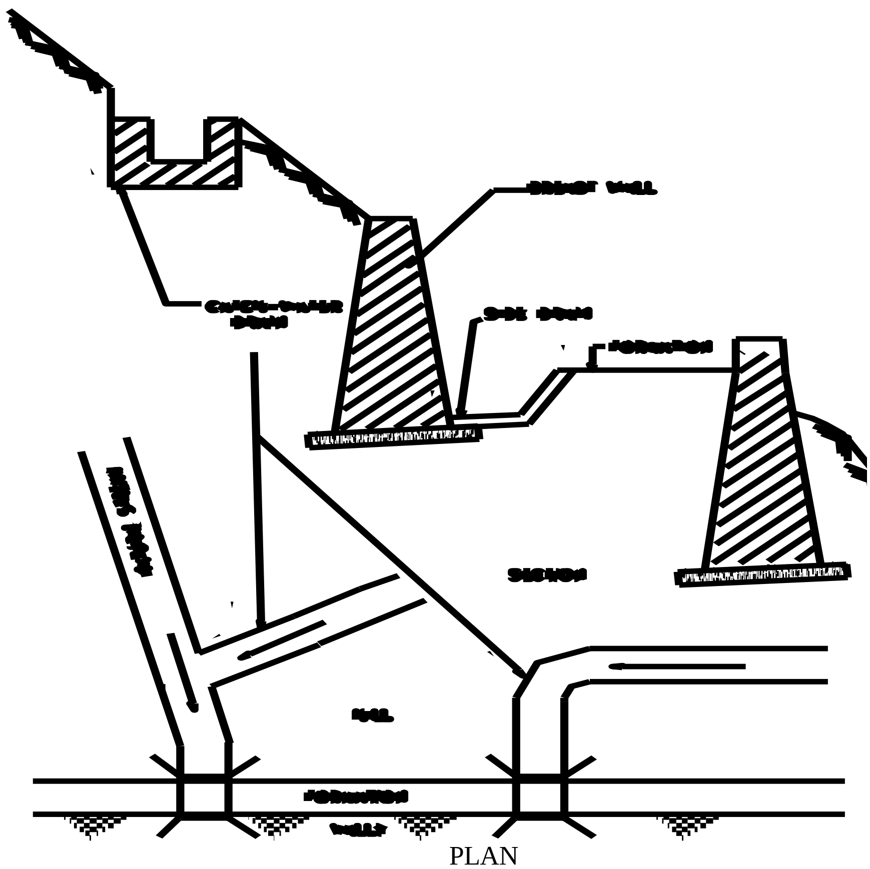


FIG. 9.4 : CATCH WATER DRAIN IN STABLE AREA

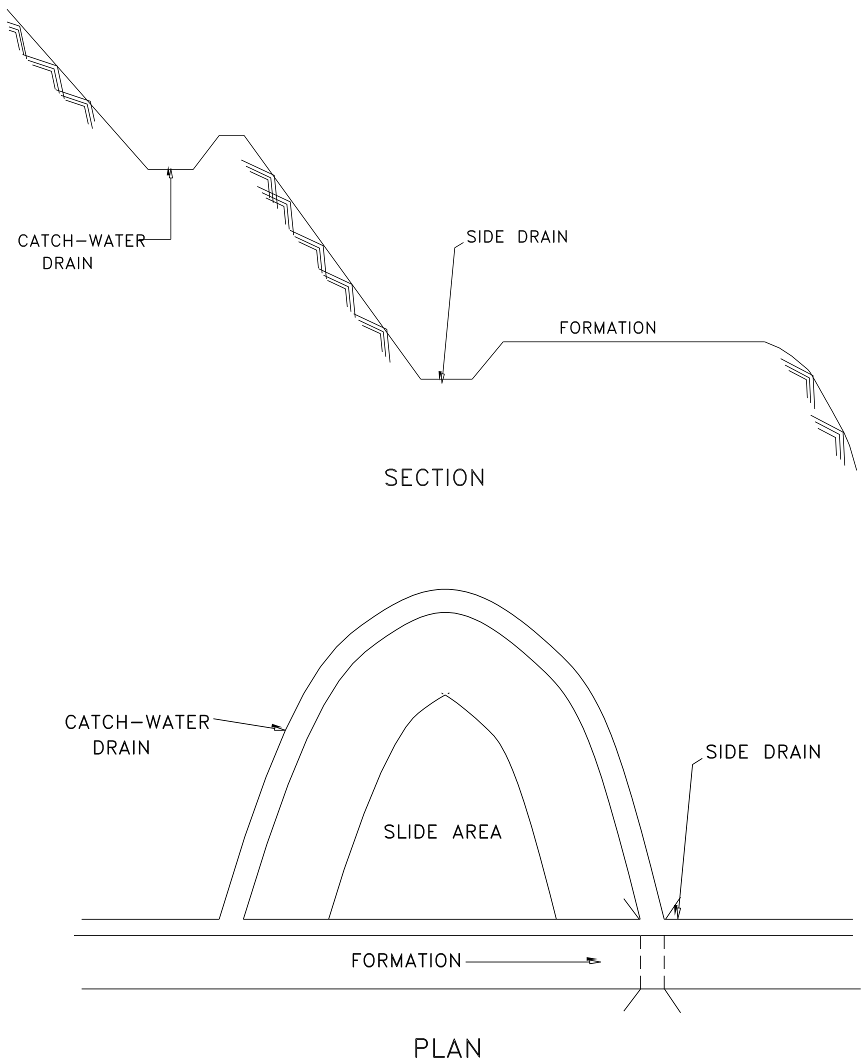
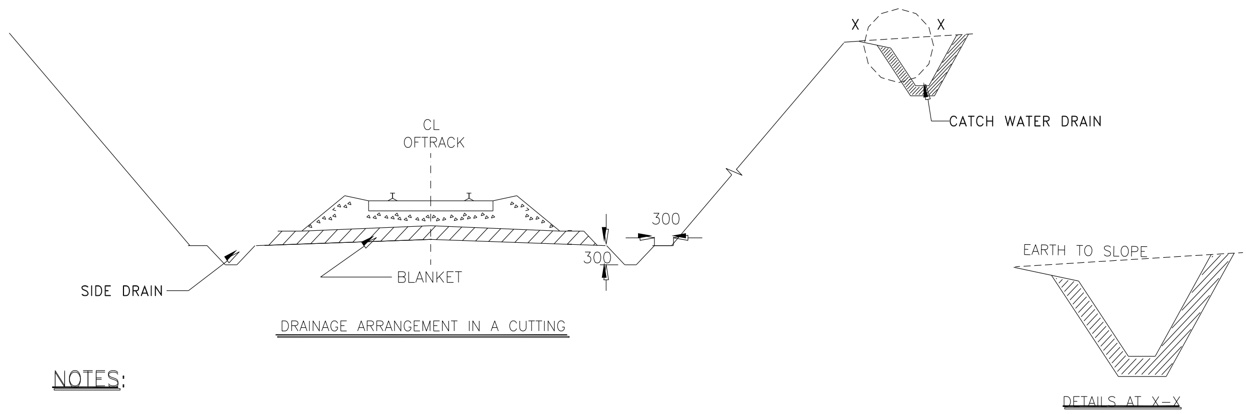


FIG. 9.5 : CATCH WATER DRAIN IN A SLIDE PRONE AREA

6.3 The arrangement of drains in cuttings has been shown in the figure below-



NOTES:

1. ALL DIMENSIONS ARE IN mm.
2. CATCH WATER DRAINS SHALL BE PROVIDED ON THE NATURAL GROUND IF FORMATION IN CUTTING IS LIKELY TO GET FLOODED FROM SURFACE WATER FLOWING ACROSS THE CUTTING OR WHEN DEPTH OF CUTTING IS MORE THAN 4m.
3. ALL CATCH WATER DRAINS SHALL BE PUCCA THE EXPANSION JOINTS SHALL BE SEALED WITH BITUMENOUS CONCRETE.
4. THE CATCH WATER DRAINS SHALL HAVE SECTION ENOUGH TO CARRY 50% MORE THAN THE REQUIRED DISCHARGE TO CATER FOR ANY BLOCKADE OR SILTING.
5. CATCH WATER DRAIN SHALL HAVE ADEQUATE SLOPE TO ENSURE DEVELOPMENT OF SELF CLEANSING VELOCITY.
6. CATCH WATER DRAIN SHALL HAVE WELL DESIGNED OUTFALL WITH PROTECTION AGAINST TAIL – END EROSION.
7. CATCH WATER DRAIN SHALL NOT HAVE ANY WEEP HOLE.

FIG. 9.6 : TYPICAL CUT SECTION WITH SIDE AND CATCH WATER DRAIN

6.4 Drainage around cracks should be given due consideration so as to prevent the build up of water pressure in them. Open tension cracks are very dangerous in areas liable to high intensity rainfall since the water forces generated by a water-filled tension crack can be very high and can induce very sudden slope failures. In addition to diverting surface water away from open tension cracks, it is advisable to prevent water from entering the cracks by sealing them with a flexible impermeable mater such as clay. When the crack is more than a few inches wide, it should be filled with gravel or waste rock before the flexible seal is placed. The purpose of this fill is to allow any water which does find its way into the crack to flow out again as freely as possible. Under no circumstances should the crack be filled with concrete or grout since this would result in the creation of an impermeable dam which could cause the build up of high water pressures in the slope.

6.5 Seepage water occurring on the face of a slope may be intercepted and carried away on benches. These Benches should be sloped back from the face and thence laterally. They should be lined so is to provide ingress of water again.

7.0 SUB-SURFACE DRAINAGE CONTROL

Only a portion of rainwater is handled by natural and man-made watercourses. The remaining water infiltrates the soil and becomes either ground water or capillary water. Where ground water is high, subsurface drainage may be needed to draw the water table down so that softening of the sub grade soils, sloughing or instability of slopes will not occur. Capillary water cannot be removed by drainage but can sometimes be controlled by lowering the water table.

The design of subdrainage installations is made from knowledge of the depth, direction of flow and seasonal fluctuations of the ground water table. Such information is best obtained by observing the soil deposits and water levels in test pits during the wet season.

Removal of sub-surface water tends to produce a more stable condition in several ways such as:

- Seepage forces are reduced.
- Shear strength is increased.
- Excess hydrostatic pressure is reduced.
- Driving forces creating instability are reduced.

The removal of water within a slope by sub-surface drainage is usually costly and difficult. Methods generally used to accomplish sub-surface drainage are the installation of horizontal drains, deep trench drains, vertical drainage wells, intercepting drains and drainage galleries. Increasing use has to be made of such techniques where high pore water pressures are the proximate cause of instability. The above techniques are briefly elaborated as under :-

7.1 Horizontal Drains

The purpose of these drains is to lower the water table and, thereby restricting water pressure to a level below that of potential failure surfaces. They are also very effective in reducing water pressures near the base of a suspected tension crack. Horizontal drains may be used in slopes where steady seepage of water is encountered. They provide channels for drainage of sub-surface water either from the sliding mass or from its source in the adjacent area. They bring about an improvement in slope stability in a short period of time.

The drain holes should be designed to extend behind the critical failure zone. The direction of the drain holes may depend to a large extent on the orientation of the critical discontinuities. Drain holes usually are inclined upward from the horizontal (say about 5^0) . In weak, soft or poorly cemented rocks, however, the holes may have to be inclined slightly downward to prevent erosion at the drain hole outlet. In this case, a small pipe can be left in the mouth of the drain hole to retard erosion. The effectiveness of drains depends on the size, permeability and orientation of the discontinuities. The optimum drain hole design is to intersect the maximum number of discontinuities for each meter of hole drilled.

The spacing and positioning of these holes depends upon the geometry of the slope and upon the structural discontinuities within the rock mass. In a hard rock slope, water is generally transmitted along joints and horizontal drains can only be effective if they intersect such features. In the case of a soft rock or soil slope, the holes can be regularly spaced but a certain amount of trial and error is necessary in order to determine the optimum spacing. In either case, the installation of piezometers before the drilling of the horizontal holes is strongly recommended since, without an indication of the change in water level, the rock slope engineer will have no idea of the effectiveness of the measures which he has implemented.

Special drilling equipment is required for installation and some periodic maintenance is needed to prevent clogging. They are often 60 m to 100 m in length, thus providing drainage deep into the slope.

Drain holes should be thoroughly cleaned of drill cuttings, mud, clay, and other materials. The drain holes not properly cleaned may have their effectiveness reduced by 75 %. High-pressure air, water, and in some instances a detergent should be used to clean drain holes. If freezing conditions exist, drain hole outlets should be protected from ice build-up that could cause blockage. Insulating materials, such as straw, sawdust, gravel, or crushed rock, have been used for this purpose.

In highly fractured ground, care should be taken to ensure that caving in does not block drain holes. If caving is significant, perforated or porous linings should be installed so that drain holes remain open. A typical sketch and arrangement of horizontal drain is given in figure below-

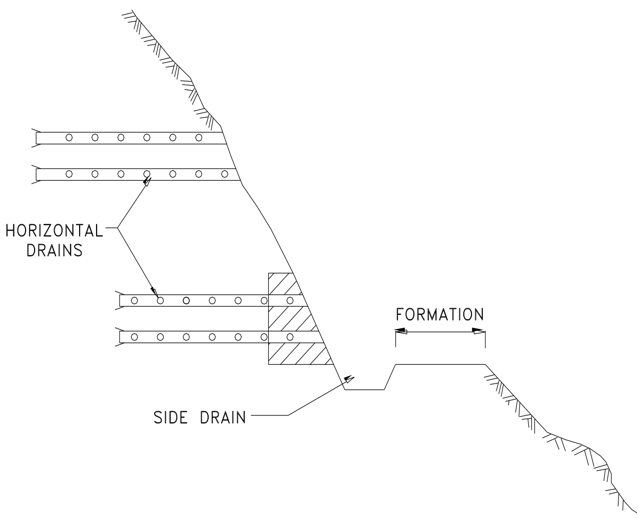


FIG. 9.7 : SLOTTED HORIZONTAL DRAIN

In nutshell-

- Horizontal drains can be used in a wide variety of soil types including weathered and fragmented rocks.
- The absence of damp spots on the rock face does not necessarily mean that unfavourable ground water conditions do not exist. Groundwater may evaporate before it becomes readily apparent on the face, particularly in dry climates.
- The maximum length of effective borings is 250 to 300 m, but at this length the end of the borehole may deviate from the intended position by several metres. This implies that it is difficult to guarantee that it will contact those aquifers in which the groundwater pressure is responsible

for impairing the slope stability. Drainage boreholes also have a limited useful life.

- For horizontal drains, perforated or slotted rigid PVC pipes can be used. Perforations or slots are generally made on the upper two thirds of the pipes. A special chuck may have to be attached at the end of each pipe to prevent its slip out from the drill hole.
- Drain holes are drilled at an outward inclination with the horizontal.
- Sub-surface drainage by horizontal drains represents a more effective solution compared to the prohibitive cost of adopting other conventional corrective measures in situations where excess hydrostatic pressure is the main cause of slope failure.
- Adequate geological and geo-technical studies should be conducted to locate the water table, determine the material properties and evaluate the benefits from horizontal drain installation.
- In critical slopes, the horizontal drains may be placed behind retaining structures like breast walls and the discharge allowed to collect by draining across the wall into side drains.

7.2 Deep Trench Drains

Deep trench drains can also be used for the purpose of sub-surface drainage. These are generally limited to those locations where water can be intercepted at depths less than 5 to 6 m. It is basically a trench where water is intended to pass through the interstices of the stones instead of a pipe. These drains are frequently plugged with fines from the adjacent ground unless protected by adequate filters. They are not recommended unless properly designed and carefully installed.

They consist of a permeable gravel core, surrounded by a filter fabric, like geotextile to prevent clogging. The gravel size is either 16-32 mm or 36-70 mm to ensure a sufficiently high void ratio.

The individual trench drains have to be made in short sections 5 to 10 long. After the trench is excavated, the filter fabric is spread out, trench is filled with gravel upto the top water bearing layer and then the fabric is overlapped. Control shafts at the junction of drains are installed in order to check the time dependent flow of water in the individual drains. Pipes may be required to feed the water into the control shafts, at some locations.

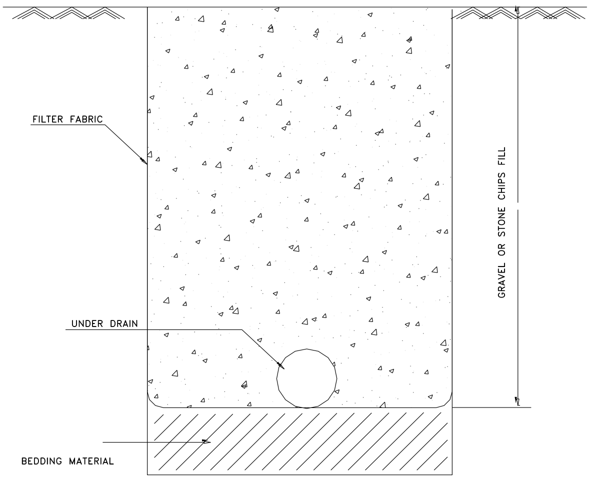


FIG. 9.8 : DEEP TRENCH DRAIN

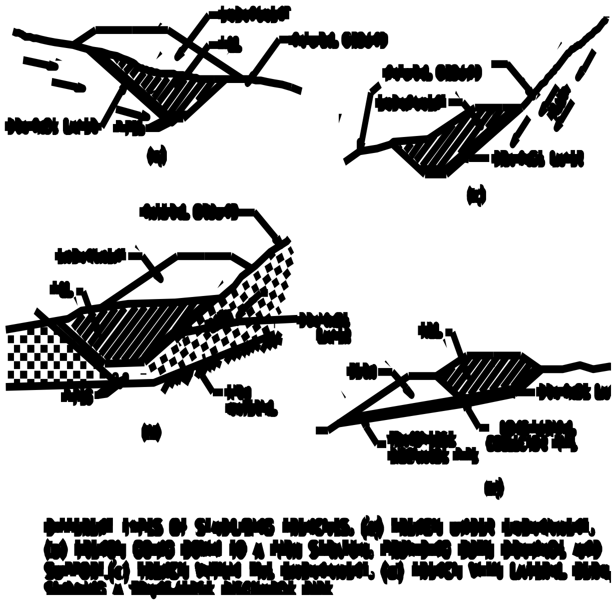


FIG. 9.9 : DRAINAGE TRENCHES

7.3 Drainage Galleries

Drainage galleries are probably the most effective means of sub-surface drainage. They are also the most expensive form of drainage and can only be considered in critical situations or where the economic benefits of steepening the slope will more than cover the cost of the drainage gallery.

Drainage galleries are conventional, deep situated structures, which are capable of discharging a large amount of water. Their principal advantage is that they permit the path taken by water percolating through the rock to be traced and thus help to establish precisely the hydro-geological conditions of the slope. The effectiveness of the gallery may be increased by making drainage borings in its walls, floor or roof. If the gallery is situated below the slide surface it may collect water from the overlying layers through these vertical boreholes. A sub-surface drainage gallery will effectively drain about 60 to 70 meters of over-lying material and, hence, for very large slopes, two or more levels of drainage galleries may be required.

In very general terms, the optimum gallery location is at the corner of a parallelogram as shown in the sketch below-

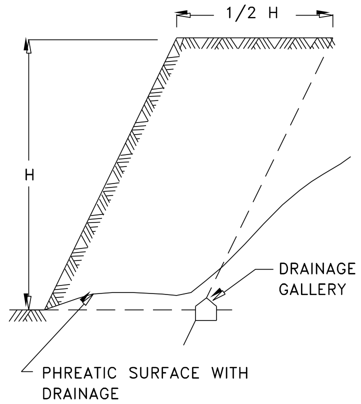


FIG. 9.10 : OPTIMUM DRAINAGE GALLERY LOCATION IN A SLOPE

7.4 Intercepting Drains

For cases, where side hill seepage under the track tends to cause subgrade softening, an investigation by auger boring or test pit should be done. To intercept the seepage before it enters the subgrade area, a side drain is provided at suitable depth as shown in the figure below so that the effect of ground water is no longer significant.

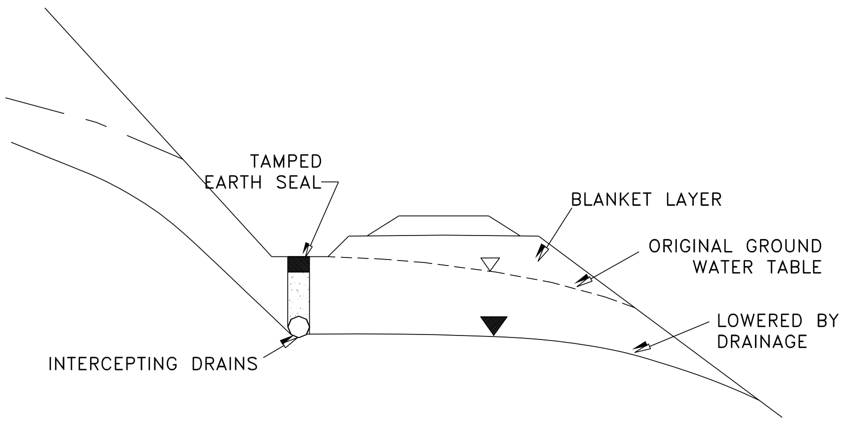


FIG. 9.11 : INTERCEPTION OF SIDEHILL SEEPAGE BY SUBDRAINAGE

Also, there may be situation where subgrade may be in the saturated condition due to high water table in a cut, in such cases, intercepting sub-drains are provided on either side of track to stabilize subgrade as shown below-

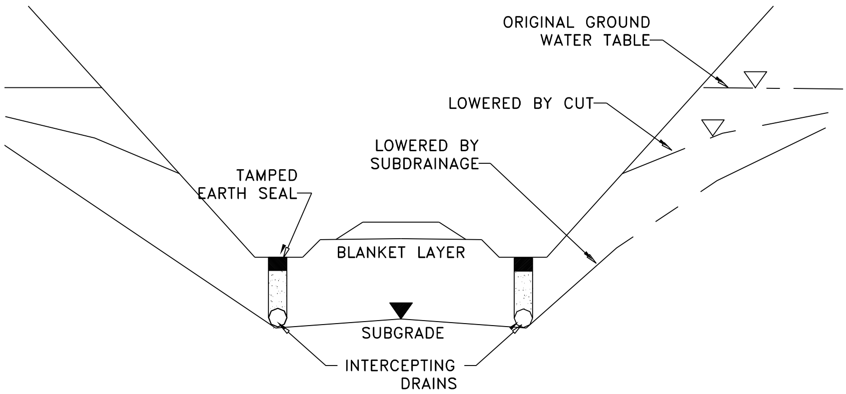


FIG. 9.12 : LOWERING OF GROUND WATER IN A WET CUT

7.5 Vertical Drainage Wells

Vertical drainage wells drilled from the slope surface and fitted with down-hole pumps can be effective in slope drainage . These wells have one major advantage in that they can be brought into operation before the slope is excavated and can play an important part in keeping the effective stresses in the rock mass high and in preventing the onset of slope movement. In rock or soil masses of low permeability, a period of a year or more may be required to depress the water level to that required for the slope design and vertical pumped wells are very effective in this type of application.

The disadvantage of vertical pumped wells is that the pumps have to be kept running for the system to remain effective. Electrical and mechanical breakdowns which could occur during periods of heavy rain are particularly dangerous if these are of sufficient duration to permit the groundwater levels in the slope to rise.

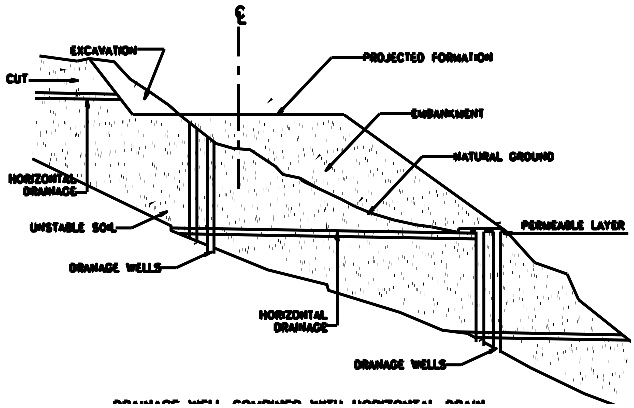


FIG. 9.13 : DRAINAGE WELLS AND HORIZONTAL DRAINS

8.0 DESIGN OF DRAINS

The drain grade will normally be governed by the track grade, particularly in long cuts. When the drain is constructed in earth materials, the minimum grade should not be less than 0.25 % to minimize sedimentation. Likewise to prevent erosion, the maximum grade should not be greater than that which will produce a velocity shown in table below. Erosion may also be prevented or reduced by paving, riprapping or constructing check dams to reduce the water velocity.

TABLE 9.1 LIMITING VELOCITIES

Material	**Velocity (m per sec)**
Coarse gravel, cobbles, soft shale	1.22 – 1.83
Clay and gravel	1.22 – 1.52
Clay	10.91 - 1.52
Grass	0.61 - 0.9
Loam	0.61 - 0.91
Sand	Up to 0.61

The drains are generally designed as open channels except when it is especially required to design them as flowing under pressure as is the case of inverted siphons and discharge lines from reservoir/catchments. Various formulas, which have been suggested for determining the velocities of flow in drains, are given below. For design of drainage, estimation of discharge can be done in a number of popular ways.

A) RATIONAL FORMULA: This is most commonly used method for calculating the run off.

$$Q = (1 / 36) \times (KiA)$$

where Q is peak run off in cumec
K is constant depending on the surface
i is rain fall intensity in cm/hr
A is area of catchment in hectares

TABLE 9.2

S.No.	**Type of surface of terrain**	**Value of 'K'**
1.	Steep bare rock	0.90
2.	Steep rock with vegetation	0.80
3.	Plateau area with light vegetation	0.70
4.	Bare stiff clayey soils	0.60
5.	Loam lightly cultivated	0.4
6.	Sandy soil, light vegetation	0.2

B) EMPIRICAL FORMULA:

a) Dicken's Formula: $Q = CM^{3/4}$, where Q is peak run off in cubic meter per second and M is catchment area in square km, C is a constant depending upon the region.

b) Ryve's Formula: $Q = CM^{\circ}$, this is applicable for Tamil Nadu Hills

c) Inglis Formula $Q = 125M/(M+10)$, this is applicable for western ghats

Use of empirical formula should be as far as possible avoided. They are primitive and are safe only in hands of an expert. The rational formula explained above is the most commonly used in the determination of discharge.

C) MANNING'S FARMULA:

$$Q = A. 1/\eta . R^{2/3}. S^{1/2}$$

Where

A	=	Cross sectional area in square metre
η	=	Co-efficient of Rugosity
R	=	Hydraulic mean depth = A/P
S	=	Bed slope
P	=	Wetted perimeter in metre

Designing an efficient trapezoidal channel

R	=	$y/2$
A	=	$y^2 \{2 (1+m^2)^{1/2} -1\}$, Tan60° = 1/m
base width b	=	$2y\{(1+m^2)^{1/2} -m \}$

Computation of non-scouring velocity mainly depends upon the material of the drains, and its values are given in table below-

TABLE 9.3

S.No.	Type of drain material	Max. Non-scouring velocity in m/sec.	Manning's η
1.	Rock	4.5-5.5	0.035-0.045
2.	R.R. masonry	4.5 -5.0	0.017-0.020
3.	Cement concrete drains	2.5 – 3.0	0.017-0.020
4.	Ordinary brick lined channels	1.5 – 2.5	0.014-0.017
5.	Earthen channels	0.6 – 1.2	0.020-0.025

9.1 The size of drains should be adequate to carry maximum water flow without disturbance to the adjacent ground or development of excessive out flow pressure. There should be no significant loss of flow by re-infiltration in to the ground along the drain length. The drain should function satisfactorily without clogging and with minimum of maintenance.

10.0 Drainage is most effective form of slope stabilization provided that the drains are properly maintained but this occurs rarely in practice. Drainage channels should be inspected on a regular basis and kept free of debris.

B. EROSION CONTROL

1.0 Slopes in soft rock or soil are prone to serious erosion during heavy rain. There is further deterioration due to weathering when exposed. The protection of the surface of such slopes can be a serious problem but nevertheless essential. The vegetation cover is almost certainly the best form of slope protection, particularly against erosion of soil slopes. A grass mat covering the slope will not only bind the surface material together but it will also tend to inhibit the entry of water into the slope. Establishing the grass cover or other vegetation is the most difficult problem since the rain which is necessary to promote the growth of the young plants is also the agent which will remove these plants by erosion.

Deforestation increases the erosion proneness and helps in mass movement in the shape of a landslide. The debris carried away by the flowing water may damage the slopes down hills and choke the streams. Thus the slope degradation by surface erosion has a multiplier effect. The amount of rainfall, soil type, slope condition and type of vegetation are major factors determining the erodibility of a slope.

Majority of slope stability problems in hill areas have their origin in cumulative erosion of hill slope. It is more economical to control the damage at the initial stage itself i.e. when it exists as surface erosion. Plantation of grass and shrubs to restore the vegetative cover has been found to be successful in arresting erosion and subsequently avoiding slope failures. Growth of vegetative covers and spread of root network to an approximate depth of 0.50 to 1.0 m helps to improve the over all stability of slope as brought out by field experiments carried out on different hill slopes for erosion control. *Various erosion control methods for slopes can be adopted as mentioned in revised " Guidelines for Earthwork in Railway Projects 2003".*

2.0 Erosion control measures are commonly classified in following categories:

a) Conventional non-agronomical system,
b) Bio-technical system,
c) Engineering system,
d) Non- conventional hydro-seeding system.

Most common methods used are the Bio-technical and Engineering System. However, appropriate method needs to be decided depending on site conditions. These methods are explained in following paras

2.1 Conventional non-agronomical system

This system uses asphalting, cement stabilization, pitching etc. This method is best utilized against seepage, erosion by wave action etc.

2.2 Bio- Technical Solution

In this system, vegetation is provided on exposed slopes. It is suited for soil with some clay fraction. Method consists of preparing slope area by grading it for sowing seeds or planting root strips of locally available creeping grass. It's root goes upto 50 to 75 mm deep into the slopes serving as a soil anchor and offering added resistance to erosion. Some typical species of grass which develop good network of roots and considered suitable are listed below:

- Doob grass
- Chloris gyne
- Iponea gorneas (Bacharum Booti)
- Casuariva and goat foot creepers etc.
- Vetiver grass (vetiveria zizanioides)

2.3 Engineering System

In this system, three methods, as mentioned below, are normally used.

2.3.1 **Geo-jute :** In this system, geo-jute is used. The system is used in areas having high erosion problems. Geojute is eco-friendly material made of jute yarn with a coarse open mesh structure and is biodegradable. On degradation, it helps in growth of vegetation. It is of two types i.e. fast biodegradable and slow biodegradable. The methodology by which geojute on slopes of banks/cuttings should be provided is explained as follows:

- Top 50 to 75 mm soil should be made free of clods, rubbish, large stones etc.
- Top surface should be properly dressed.
- Seeding should be done by distributing evenly over the slope.
- Folded geojute should be buried at critical slippage of top soil.
- Geojute is then unrolled loosely and evenly.
- Up channel and shoulder are buried and stapled and then anchored as per the requirement of the supplier.
- Down channel ends and toes are folded and secured as per manufacturer's requirement.
- Wherever it is getting terminated, it should be buried as per specification.
- Longitudinal edge overlapping should be as per manufacturer's requirement and stapled at one meter interval.

- Rolled junctions are overlapped as per manufacturer's requirement.
- Up channel section over down channel additional row of staples is fixed at 1m interval down each strip.

Watering facility should be ensured during initial period of sowing if the work is undertaken during non-monsoon period. Post laying protection against stamping and grazing by cattle is required. In case of any damage, local spot repair should be carried out. Once vegetation is well established, no maintenance is required.

2.3.2 **Polymer Geogrids:** Under unfavorable soil and rainfall conditions where vegetative growth is difficult or is considered inadequate, a synthetic root reinforcement vegetation system using geogrids should be provided. Geogrids are flexible, non-biodegradable, and resistant to chemical effects, protected against ultraviolet degradation and are stable over a temperature of 60-100 ˚C. It provides root matrix reinforcement with dense vegetation growth that works as permanent measure against erosion. Simply extruded un-oriented and unstretched polymeric grids of low mass are considered adequate. In deep open cuttings with boulder studded strata, bi-axially oriented geogrids of low mass should be deployed with suitable anchors to retain the boulders in place till growth of vegetation is adequate. Following methodology should be adopted for laying at site.

Slope area should be dressed with filling of cavities and pot holes if any by light ramming. The net should be unrolled ensuring uniform surface contact. Geogrid ends at top and bottom of slopes should be suitably anchored in 50cm x 50cm size trenches. MS pins 6mm dia and 2m c/c should be used to hold the lightweight net in position for an initial period of 2 to 3 months. Overlapping of grids (about 75mm) and jointing with 6mm dia, ultraviolet stabilized polymer braids is required to be carried out in longitudinal direction of laying. No overlaps are required in transverse direction of laying while jointing.

2.4 Hydro-seeding System

This is non-conventional and innovative system of development of vegetation. This system can be tried on mountainous slopes and steep banks/cuttings. In this system, Verdyol mulch solution @ 100 to 150 gm/m^2 is sprinkled on the surface for germination of vegetation depending upon the local soil and the climatic conditions.

CHAPTER VI

ROCKFALL PREVENTION, STABILZATION AND PROTECTION OF SLOPES

1.0 Identification of a weak spot, its monitoring, prevention of occurrence of a failure, its timely warning, reduction of impact in the event of failure, and availability of rehabilitative schemes are the major steps in the direction of cutting zone failure mitigation. There are several techniques available for carrying out each of these steps with due diligence. The choice of the technique will have to be exercised by the field engineers carefully.

2.0 <u>IDENTIFICATION OF PROBABLE WEAK LOCATIONS</u>

2.1 Failure Potential Evaluation Mapping (FPEM) of the hill slope areas adjoining the track should be carried out by conducting detailed study of interactions of various causative factors and sub-factors (listed in Annexure-II) responsible for contributing slides. This map indicates various segments with different degrees of failure potential.

In order to make reliable FPEM with minimum time, it is necessary to use the Geographic Information System (GIS) package. The maps produced can be quickly evaluated before and after the monsoon of every year to give the latest situation of the existing slide in the area as well as other failure prone hill slope areas. Such maps help in identifying actual landslide hazards probability under different geological, geo-technical and climatic conditions without any special training to field staff.

2.2 Probable Landslide Disaster Mitigation Map (PLDM) shows different types of suitable remedial measures at different slide prone areas as indicated in FPEM. The conventional remedial measures are modified depending upon the existing field conditions. All measurements are marked on the map to the scale.

Mitigation of various slides along the track can be successful only when thorough knowledge is obtained from the factors responsible for occurrence of disaster, their expected frequency, type or character and magnitude of sliding mass. The FPEM map of hill slope adjoining track, therefore, must be made so as to produce adequate information for taking proper remedial measures in advance for the safety of the hill track.

3.0 <u>INSTRUMENTATION & MONITORING OF SLOPES</u>

3.1 Slopes seldom fail without giving ample warning and if these warnings are heeded, the stability of potentially dangerous slopes can be improved and slope may still fulfill its function. At times, because of shear geographical extent of the workplace, monitoring of slopes through field instrumentation may not be practically feasible. However, the techniques of field instrumentation may be useful if the railway track is passing through an active landslide zone or effect of some nearby construction activity on a cutting is to be seen. Field instrumentation and monitoring is required for detecting signs of impending instability as well as post-slide movements. Observational data on vertical and lateral surface and sub-surface movements, and piezometric pressures within the unstable slopes are necessary for evaluation of stability and design of control measures.

Typical situations for which various instruments have been used are the following:

- Determination of depth and shape of sliding mass
- Determination of absolute lateral and vertical movement within a sliding mass
- Determination of the rate of sliding
- Monitoring of the activity of marginally stable natural or cut slopes and identification of effect of construction activity or precipitation
- Monitoring and evaluation of the effectiveness of various control measures

The instrumentation programme should provide the basic information of the following aspects.

- Measurement of surface movements
- Measurement of sub-surface movements
- Monitoring of the build-up and dissipation of pore water pressure at different points in the slide area, specially around the failure plane.

3.2 Monitoring of Surface Movements

Surface movements may be horizontal or vertical. To measure the subsidence or horizontal surface movement, pegs or surface markers are fixed at a number of predetermined points on the surface. All the measurements are taken from a fixed observation post or permanent bench marks which are installed at a place not likely to have any disturbance and is located away from the zone of slide.

3.2.1 The opening of tension cracks is usually the first sign that a slope is in distress. The monitoring of the movement of such cracks usually gives a very good indication of the overall behaviour of the slope. In a typical tension crack monitoring system, four pegs are set into holes drilled into the rock on either side of the tension crack. Epoxy resin is used to cement the pegs into the holes. A large Vernier Caliper can be used to measure the displacement across a wide tension crack. For narrow tension crack, a mechanical extensometer can be used. Also, a precision level can be used to determine the changes in the level of the pegs. To detect shear movement along the tension crack, measurements are made across the diognals as well as along the sides of pegs driven.

Other common system as shown in the figure below utilizes a wire anchored to the front block and running over a pulley which is fixed on the block of rock behind the tension crack. The movement of a weight suspended on the end of the wire gives an indication of tension crack movement.

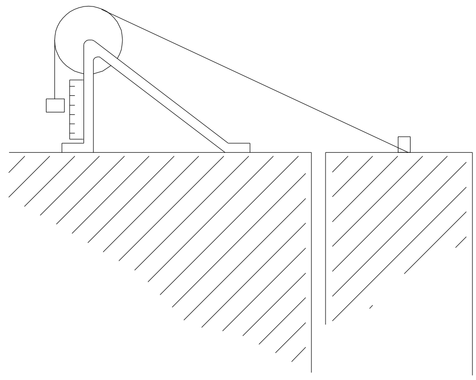


FIG 1 :   A SIMPLE TENSION CRACK MONITORING SYSTEM

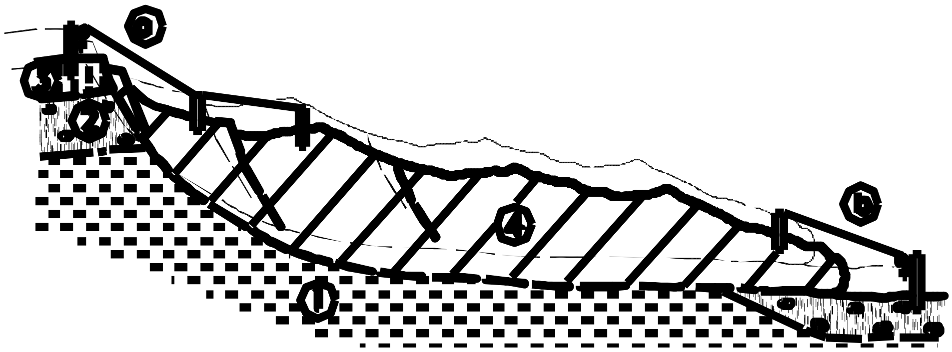
3.2.2 In conventional methods commonly applied for monitoring slide movement, a network of measuring points is laid out on the surface of the slide area, and their displacement relative to fixed points located on the adjacent stable area is measured. The monitoring is carried out either at regular intervals or at times depending on the factors controlling the rate of movement, for example, rainy periods and earthquake. These methods are satisfactory where the rate of movement per year is greater than 2.5 to 5 mm, which is generally the error of measurement.

3.2.3 For moving rock slopes, monitoring system generally involves the installation of a number of corner-cube reflecting targets on part of a moving rock slope and the monitoring of the movement of these targets with an electro-optical distance measuring device. If the targets are well placed and if they are monitored regularly, a contour plan of slope displacement rates can be prepared which will assist in defining the extent and the critical portions of the slide. A careful examination of movement often provides a valuable insight into the mechanism of failure.

3.2.4 Electronic distance measurement (EDM) devices make possible direct measurement of horizontal distances by means of electromagnetic waves; laser beams may also be used. This technique gives more accurate results in a much shorter time. The accuracy of this method is to some extent affected by weather conditions.

3.2.5 Another method usable for movement measurement is the photogrammetric approach. Sequences of photographs taken simultaneously from two or more permanent sites allow the displacement to be determined.

3.2.6 A direct measurement of slope movement can also be conducted with the use of simple wire extensometers as shown in the figure below-



(1) TERTIARY CLAY (2) SAND AND GRAVEL (3) LOESS LOAM (4) SLIPPED MATERIAL

FIG. 2 : WIRE EXTENSOMETER (a,b)

In order to establish the displacement, a point on the landslide body is connected by a wire with a benchmark on the stable terrain. Readings are taken mechanically or are registered electrically.

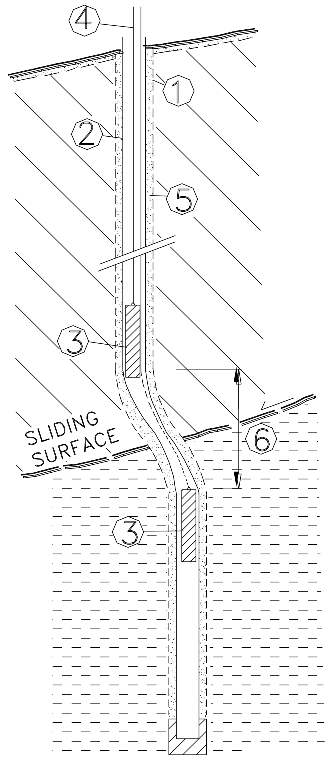
3.2.7 Dilatometers, which measure displacements in fissures of solid rocks, are based on mechanical, electrical or other principals. A dilatometer can be used for the measurement of very slow movements. Readings can be made either visually or photographically. It is possible to detect very slow slope movements of the order of 0.1 mm/yr despite temperature interference.

3.3 Depth of Slide Surface and Monitoring Sub-surface Movement

Normally, the sliding surface can not be observed visually nor it is apparent from surface measurements. The knowledge of actual deep seated failure plane is of use for properly understanding the mechanism of sliding. The data is also of utility in designing remedial measures.

3.3.1 For this purpose, test pits, test trenches and borings are employed. Pits and trenches are advantageous in that they permit direct inspection of the individual beds and easy taking of undisturbed samples for laboratory research. They are usually dug in slides that are already at rest. If a borehole strikes several slide surfaces in succession, it is necessary to be certain that the deepest slip surface has been found.

3.3.2 The depth of the slide surface in an active landslide can be established from the deflection of the borehole at the level of this surface. An active slide surface can be located by sinking two lead mandrels in a borehole. A thin-walled plastic tube (about 15 mm across) is set on the bottom of the borehole and sand is packed around it. One mandrel is lowered on a wire into the tube and the other is passed down for each measurement as shown in the figure below.



(1) Borehole
(2) Plastic protecting tube
(3) Measuring mandrel
(4) Wires
(5) Sandy backfill
(6) Bending of the tube

FIG. 3 : MEASUREMENT OF DEPTH OF SLIDE SURFACE

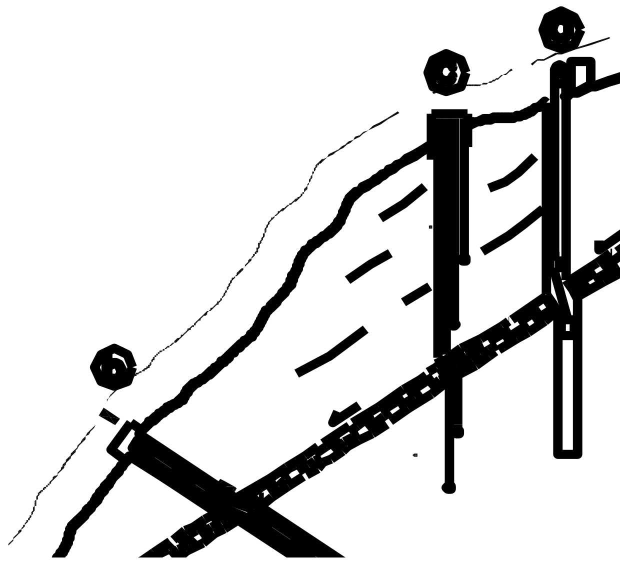
On the renewal of the movement, the tube bends at the slip surface and the mandrels cannot pass through it. A disadvantage of this device is that it does not give any indication of the direction of the movement.

3.3.3 For giving indication of direction of movement, inclinometers of several types are used. For inclinometer measurement, holes of a larger diameter (up to 100 mm) are bored so that an inclinometer used for measuring the curvature of borings can be inserted. Precise orientation of the inclinometer inside the tube can be known. It is possible to register any deflection from the vertical in two perpendicular directions. Measurement can be taken over the entire length of the tube and a continuous profile can be drawn of the deflected hole, unless the tube is sheared off by the slide.

Results of high accuracy are obtainable with an instrument referred to as a ‘digital inclinometer’ indicated as ‘a’ in figure below; it makes it possible to monitor movements that are perpendicular or oblique to the borehole axis.

For determination of an active slide surface, ‘multipoint deflectometer’ indicated as ‘b’ in figure below, which combines the principle of the inclinometer with that of the extensometer can be used.

To determine the depth of a slide surface and the rate of movement, a 'shear-plane indicator' shown as 'c' in figure below, can be used. It consists of several wires positioned at different depths in the borehole, particularly where the active slide surface is expected to be. If movement occurs along a surface penetrated by the borehole, all wires below it will show a double bending, whereas those above it will remain straight. As a result of this bending, the upper free ends of wires are pulled into the borehole, and the extension of the wires is easily measured at the ground surface. Various arrangements are shown in the figure below-



(a) Digital inclinometer,
(b) Multipoint deflectometer,
(c) Shear-plane indicator

FIG. 4 : VARIOUS SYSTEM OF SLOPE MOVEMENT MONITORING

3.4 Geophysical Methods

Geophysical methods may be used to complement the results of direct investigation.

3.4.1 The presence of weak layers (loosened rock masses or faults) can be traced by electrical resistivity and by seismic methods; the loosened material displays lower resistivity and a lower velocity of seismic wave transmission. The resistivity method is most promising in the higher parts of slopes, where there is a smaller possibility of groundwater interference.

3.4.2 The position of the slip surface and the rate of movement can also be determined by the geo-acoustic method. The principle of this method consists in quantitative registration of sub-audible emissions arising during the disturbance of the rock by a slope movement. Rock noise is scanned by sensitive electro-dynamic geophones and is recorded on tape. For the purpose of registration, specially cased boreholes are used. The records are then evaluated with oscillographs. Because other sources of acoustic noise are present in soil and rocks, a problem of interpretation often arises.

3.5 Monitoring of Pore Water Pressure

3.5.1 Monitoring pore pressure is essential for effective stress analysis of the slide prone area. Pore pressure measurements are required around the failure plans for control measures to be adopted as well as for mitigating potential slides. Groundwater levels and pore pressures in a landslide area can be measured by piezometers. Piezometric heads should be plotted on graphs showing amount of rainfall and other data that may influence the pore-pressure. If drainage system has been provided, the quantity of seepage should also be recorded and plotted.

3.5.2 When making piezometric pressure measurements, it is essential that the sections of borehole in which the sensor is located is adequately sealed from water flows up or down the borehole. If this is not done, the pressure recorded will not necessarily be the piezometric pressure existent in the rock about the sensor.

Many types of rock require large flows of water to activate the piezometer. In such cases, the response time will be very slow and it is advisable to use one of the many piezometer types which do not require appreciable flows of water into or out of the sensor.

3.6 Common Type of Instruments

3.6.1 Extensometers

Extensometers measure the increase in the length or extension of a wire or rod connecting two points. They may be either surface mounted or installed in boreholes. While extensometers are based on a number of principles, the most widely used are mechanical devices employing rods, wires or tapes etc.

Tape extensometers commonly consist of metal tapes with a tensioning device permitting the tape to be uniformly tensioned between fixed anchor points. The tensioning device may incorporate a dial gauge permitting readings of relative displacement. Temperature corrections may be required. They find extensive application in the measurement of surface movements.

Rod and wire type extensometers may be either surface mounted or installed in boreholes. They both operate on essentially the similar mechanical principles, but in the case of the wire type extensometer, the wire extension elements are tensioned between the anchor point and the reading head. Wire extensometers are usually less expensive than rod types and a large number of anchor points can be installed in a comparatively small diameter drill hole. They are, however, more likely to be affected by friction effects along the length of the borehole and by corrosion. Both varieties may be equipped for either manual or remote readout arrangements.

3.6.2 <u>Inclinometers</u>

They work on the principle of change in inclination by determining the change in slope at various points; integration of the slope changes between any two points yields the relative deflection between those points.

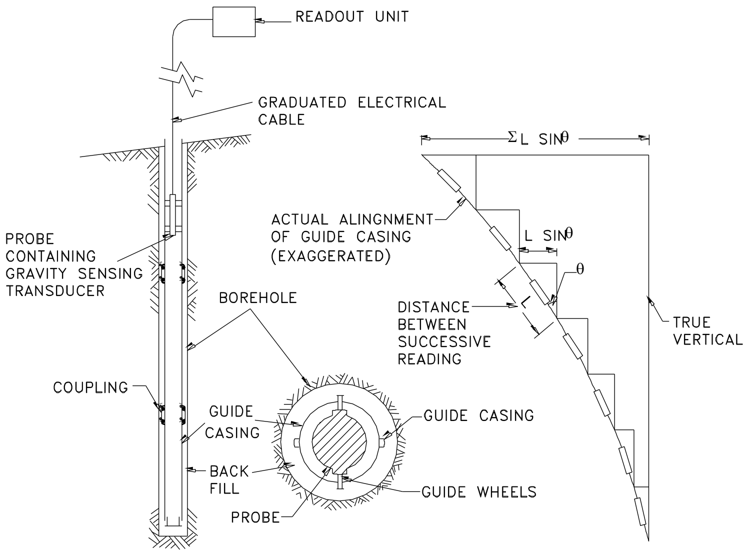


FIG. 5 : INCLINOMETER

Most of the inclinometers have a probe containing gravity sensing transducers which measure inclination of the pipe with vertical. Horizontal movement of the pipe with respect to its vertical alignment is computed from the data, provided by the inclinometer. Special pipe which is grooved in the longitudinal directions is placed in near vertical position in the borehole and the probe travels vertically downwards. Measurements are taken at regular intervals of depth with a readout unit. Though a variety of inclinometers are available in the market, essentially all of them have four similar features:

- A permanent guide casing installed in a near vertical alignment.
- A portable probe containing a gravity sensing transducer.
- Portable readout unit for power supply and indication of probe inclination.
- Graduate electrical cable linking the probe to the readout unit.

3.6.3 <u>Piezometers</u>

A large variety of piezometers are commercially available and are classified as follows:

- Hydraulic piezometers
- Pneumatic piezometers
- Electrical piezometers

Hydraulic Piezometer

A piezometer, which reads the water pressure through rise in water level in its standpipe or through a pressure gauge is called a hydraulic piezometer. They are cheap and effective, but require substantial flows.

This simplest type of hydraulic piezometer is the Casagrande open stand pipe type piezometer. It is a low cost instrument and is available indigenously. However, it does not depict changes in pore pressure immediately. Because of low cost, ruggedness, simplicity in operation and easy availability, the Casagrande type of piezometer is extensively used.

Casagrande open stand pipe piezometer consists of a ceramic porous tip connected to open stand pipes. The ceramic tip is generally of low air entry value which exhibits very high water permeability. The piezometer is installed in a cased borehole and shrouded with sand. Depending upon the pore water pressure existing upon the porous tip, water would rise in the standpipe until the hydrostatic head of the column of water in the stand pipe is equal to the pore water pressure. The height of water in the stand pipe may be measured with an electronic water sensor. In order to record a given incremental pore water pressure within the ambient soil, large volume of water is required to flow into the piezometer unit.

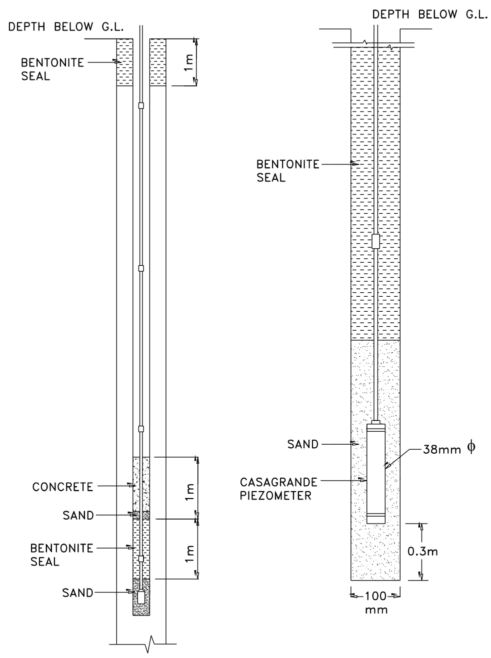


FIG. 6 : DETAILS OF OPEN STAND PIPE PIEZOMETER (CASAGRANDE TYPE)

Pneumatic Piezometers

The pneumatic piezometers are difficult to operate and have many disadvantages, such as, regulating the gas pressure at par with pore pressure in sub-soil and detective the outflow of gas for pore pressure indication.

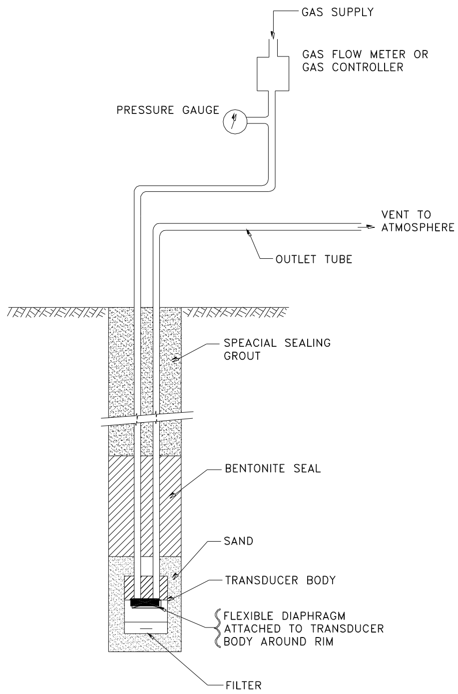


FIG. 7 : PNEUMATIC PIEZOMETER INSTALLED IN A BOREHOLE

Electrical Piezometers

The electrical piezometer mostly contain a vibrating wire transducer on which water pressure acts through a ceramic tip. It is easy to install and responds quickly to pore pressure. Whereas hydraulic piezometers, especially the Casagrande type is installed in a borehole, at a particular depth and the piezometer tip is sealed to prevent surface water from interfering with the pore water, electrical piezometers can be pushed in the soil with special equipment to the desired depth. The electrical piezometer is required to be calibrated before its installation.

4.0 METHODS OF WARNING FOR DETECTING ROCK FALLS

Warning methods have been used on railways in mountains to detect rock falls on tracks so that trains can be stopped before hitting the material. They are necessary where other measures are too expensive or impractical or where a new hazard has developed. The types of warning system, which are in common use, are following:

i) **Patrolling Gangs**

A patrolling gang is deputed on vulnerable locations specially in monsoon period as in this period risk of falling of boulders on track is quite high. Frequency of patrolling can be adjusted to the demand of weather and traffic conditions and also on vulnerability of the locations. This method is simple and reliable and flexible.

ii) **Electric fences**

A falling rock mass capable of endangering traffic will break or pull out one of the wires and thus actuate a signal to warn approaching traffic. Based on above principles, a system consisting of a row of poles spaced along the up hill ditch line and wires strung between them at a vertical spacing of about 250 mm is provided.

iii) **Wires**

In this system a single wire is anchored at both the ends and linked to a warning signal device. Such a wire may be fastened around a large unstable rock or across a rock slope above track where probability of large rocks rolling down is high.

iv) **Instrumentation Warning**

Inclinometers, extensometers and piezometers can be coupled with alarm buzzers. These instruments are pre-calibrated to sound alarm signals at a predetermined rate of movement or increase in a particular parameter.

5.0 STABILIZATION AND SURFACE TREATMENT

Stabilization methods may be grouped as under

- Excavation - to remove unstable material
- Drainage - to reduce internal water pressure
- Surface treatment - to retard weathering
- Support & restraint system - to strengthen rock masses
- Damage control & impact reduction system - to contain the damage

These are explained in detail as under:

5.1 Excavation

Removal of loose rock from faces and slopes is a basic maintenance operation**.** Also**,** removal of overburden pressure from the top of the cutting, where cut-spoils might have been dumped is a very good stress relief technique.

5.1.1 Loose Scaling and Trimming

Scaling refers to the removal of loose, overhanging or protruding blocks using hand bars, hydraulic splitters and explosives. Trimming involves drilling, blasting and scaling to remove small ragged areas of rock where repetitive scaling would otherwise be required. Success and economy of scaling and trimming depends upon

- Type of equipment used
- Efficient planning with safety requirements
- Decisions to be taken based on rock condition

Loosely held back rock, sub-rounded boulders mixed in soil matrix at the edge of the cutting & shattered rock mass with open joints/cracks over the slopes can be removed after thorough inspection of each location. However, even after extensive loose scaling, the rock/soil strata over the cutting slopes may tend to get loose, due to various factors like, weathering effect, seepage along weak planes, erosion of soil around sub-rounded boulders in heavy rains, vibrations, undermining due to erosion etc. It is therefore, essential to carry out checks at regular intervals. It shall be preferable to have special gangs to carry out loose scaling work periodically.

5.1.2 Removal of Overhangs

In some rock cuttings, overhangs in top section of cutting slope may be noticed sometimes due to undermining of soft zones. Such overhanging rock masses are vulnerable and tend to get detached. These overhangs should be either removed by extensive loose scaling or by controlled blasting.

5.1.3 Flattening of Slopes & Creation of Berms

In high depth cuttings with fairly steep slopes, wide berms as already detailed in previous chapter are to be provided at appropriate levels to increase the stability of slope along with flattening of slopes, if required. In soil-rock cuttings which have thick soil mantle/cover in top section & fairly steep soil slopes, it is desirable to provide 5 m wide berms at soil-rock interface.

5.1.4 Controlled Blasting

In very deep rock cuttings, having sub-vertical slopes, highly jointed/fractured strata, overhangs and repeated history of boulder fall in every monsoon, controlled blasting may provide the permanent solution. In such situations, the controlled blasting can be done with the help of specialized agency to remove the overburden by flattening the slopes suitably and creating berms in rock strata at every 6 m to 7 m. Drilling of holes and loading with explosives is to be done in such a manner so that the rock debris and muck does not move down towards the track after the blast.

5.2 Drainage

Apart from erosion protection, reduction of hydrostatic pressure is a major drainage function that helps stability of cuttings. Improvement of surface as well as sub-surface drainage helps in release of pore pressure. Various methods and techniques for improvement of surface and subsurface drainage have already been discussed in Chapter.V.

In impermeable soils, stabilization of slopes by drainage often fails since the area of zone that is well drained is small. In such cases, the method of soil hardening by electro-osmosis may be applied. This method differs from sub-surface drainage in that the water does not move under the influence of gravity but is acted on by an imposed electric field. If a potential difference is set up between two electrodes placed in soil, water migrates towards the cathode. This is a perforated pipe, from which water can be removed by pumping.

5.3 Surface Treatment

5.3.1 Shotcreting

Slopes prone to spalling, rock falls & sliding of small volumes of rocks can be stabilized effectively by spraying the face with concrete. Shotcreting is used to prevent weathering of rock surfaces and to provide structural support. It is a mixture of sand, cement & aggregate upto 12 mm size which is spread on the rock surface with the help of shotcrete equipment. When shotcrete is spread on the highly jointed/fractured/weathered rock surface, it fills the openings like cracks, fissures, open joints & covers the joint planes/weathered rock surfaces & helps in minimizing rock displacement. However, weep holes are provided so as to ensure drainage of water from the rock mass.

The shotcrete is very useful in rock strata which tends to get weathered fast & causes its undermining. It is normally applied in layers upto100 mm thick. When these materials are applied to an irregular rock surface, the resulting surface configuration is smoother. The sprayed concrete helps to maintain the adjacent rock blocks in place by means of its bond to the rock and its initial shear and tensile strength acting as a membrane. The result is that a composite rock-concrete structure is developed on the surface of the rock. There is no transfer of load from the rock mass to the sprayed concrete lining. The interlocking quality of surface blocks is improved, the sprayed concrete acts as reinforcement and not as support. The more quickly the concrete is applied after excavation, the more effective results are.

Deterioration of sprayed concrete can result from frost action, ground water seepage, or rock spalling due to lack of bond with the underlying rock. It is therefore important that, prior to spraying with concrete, the face should be thoroughly scaled and trimmed and any unfavorable ground water flows should be drained to provide long term stability. Weep holes should be drilled or installed through the hardened sprayed concrete lining and into the rock for free flow of seepage/underground water and to prevent development of water pressure behind the face. Short flexible plastic pipes are placed in cracks or holes drilled into water bearing broken rock. Sprayed concrete can be used in combination with steel wire mesh and rock bolts to give structural support and also to form buttresses for small loads. Where it is applied to mesh, all loose material should be removed from the rock surface and the mesh fabric should be tightened

Following general points may be kept in view while carrying out shotcreting work-

- Before starting the work, rock surface shall be cleaned with water and compressed air to remove clay, mud etc. existing on discontinuities and to ensure a good bond between the concrete and rock.
- Weak material should be removed before starting the work.
- Cement bags older than three months from the date of manufacturing shall not be used under any circumstances.
- Layer of shotcrete shall be systematically sounded with a hammer to check the dummy area.
- Weepholes of maximum 38 mm diameter at appropriate spacings shall be provided in shotcrete.

5.3.2 Grouting

The term grouting is used to describe the injection of cement suspensions under pressure into voids or cracks in massive rock. The primary requirement of a grout are

- that it be sufficiently fine and viscous to penetrate the finest capillary openings in the rock
- that following placement, it solidifies or gels sufficiently to fill all voids and fissures with a solid material, thus both increasing the strength of the rock and preventing influx of water.

The most widely used grouting material in rock treatment is Portland cement with additives designed to improve its viscosity and act as filling material, while at the same time retaining a high setting strength. Common additives are montmorillonite clays (bentonite), sodium silicate to improve viscosity, and flyash or fine sand as a filling material. Addition of either of these additives in large quantities can weaken the strength of the grout, except in the case of fillers such as flyash suspensions and sodium silicate mixtures with their intrinsic ability to gel on settlement can be used where an extremely fine and free-flowing grout is required for the primary purpose of reducing the permeability of a porous rock.

Injection grouting as a treatment for stabilizing in soft cuttings has been experimented over years and has been found suitable for railway cuttings. A grid of injection point is set up with the base of the points below the slip plane, and a quantity of grout is specified for injection into the various points based upon experience, the local conditions, and depth of injection.

The distances through which the cement grout can be forced through minute cracks depend upon its viscosity or thickness. The later varies as per size of the crack. More than one level of injection may be required. It is also suggested to do some experimentation at the start of operation, as it is required to determine the proper mixture.

The pressure at which grout may be pumped into rock strata depends on the degree of confinement, the strength of the rock and the presence of large scale discontinuities, factors which may combine to cause failure in the rock when subject to the hydraulic pressures associated with grouting. With this reservation, the higher the pressure the more effective will be the penetration, spread and strength of the grout. In places where rock is particularly weak, containing numerous voids, high grouting pressures which might initially fracture the rock can be obtained through repeat injections after voids have been filled and the rock stabilized by low-pressure injections of coarse grout.

Grouting can therefore be used to improve rock properties in two ways; through the reduction of water content by increased permeability and reduced porosity of a rock, and through consolidation of weak and fractured rock masses by bonding of cracks and weakness planes.

5.3.3 Stone/Boulder Pitching Soil Slopes

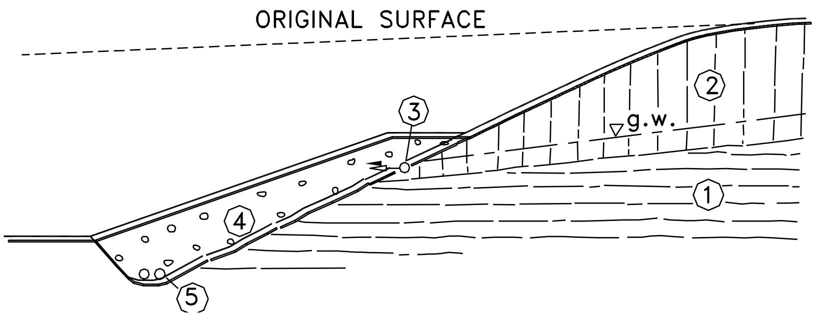
Pitching is a covering of hard material like stones, boulders over loose/soft soil slopes strata to prevent its erosion in rainy season. Rough stones are commonly used for pitching with a thickness varying from 25 cm to 60 cm. The pitching stones should be the heaviest available that can be handled and roughly cut to fit in properly. The stones should be tightly hand packed and laid with their broadest face downward and at right angle to the slope. All interstices, hollows between the stones, should be filled with smaller pieces. The outer face of the pitching should be made as smooth as possible, so as to prevent scouring lower down, by way of cement mortar, printing and any other means.

5.4 Support and Restraint System

5.4.1 Buttresses/ Counterweight Fills

Counterweight fills provide sufficient dead weight near the toe of the unstable slope to prevent the movement. These provide an additional resisting component thereby increasing the factor of safety against failure. The counterweight fill should be placed on a stable foundation layer with adequate depth. These are built at the toe of landslides with materials such as gravel or crushed stone. Their effectiveness may be further ensured by the provision of drainage structures.

These must be designed to resist the driving forces, i.e. overturning, shearing and sliding at or below the base. Stability analysis can be carried out using standard procedures available. It must also be ensured that the counterweight fill itself is stable during as well as after the construction.



(1) Clay (tertiary), (2) Loess loam, (3) Spring, (4) Sand and gravel, (5) Drainage

FIG. 8 : GRAVEL BUTTRESS

5.4.1 <u>Retaining walls</u>

These are used to prevent large blocks in the slope from failing and to control failure by increasing the resistance to slope movement and bring greater stability to slopes. They have proved suitable where the ground is free-draining. Otherwise, pervious backfill, drain pipes, etc., have to be proved.

Various types of retaining walls, viz. PCC/RCC walls have been provided, as a toe support to the soil slopes. In the design of these retaining walls, the thrust on back of the wall due to earth pressure is taken into consideration and the wall is constructed with required thickness and weight to resist the movement of soil mass from behind. Backfilling of the walls is done with granular materials or small stones to decrease back pressure on walls and help drainage through weep holes provided in the walls.

5.4.3 <u>Gabions</u>

Gabion retaining walls are mass gravity structures that are often used because of their economic, functional, and aesthetic characteristics. These characteristics combined with other advantages, such as their flexibility and permeability; make them a good alternative to other types of retaining structures. The design principles are simple and rudimentary.

Gabions provide a versatile method of constructing retaining walls speedily and economically. Gabion is a basically a rectangular box strongly made up of steel wire-mesh which can be assembled at site and filled with hard local boulders/stones. The gabion boxes come in several sizes of 2 m to 4 m length, 0.3 m to 1 m width and 1 m in height. The steel wire-mesh of gabion is square or hexagonal, doubly twisted and galvanised/PVC coated to prevent its corrosion. The assembled gabions can be tied together, braced and finally placed firmly at toe of cutting slope. The process is carried out in-situ i.e. at the location where the gabions are to be installed.

Of late, gabions are also assembled from perforated geogrids. Geogrids, which are made of polypropylene, have high resistance to impact and weathering besides possessing good strength and elongation characteristics.

A row of gabion walls may be placed at the toe of the slide so that these serve to improve the stability of the slopes by their dead weight. It is a common practice to use gabion walls to serve as breast walls as well as in the middle of running slopes where they serve as check-walls. In such applications, gabion walls help to retard the flow of water and reduce the surface erosion of the slope, to a certain extent. Figure given below illustrates the gabion wall-

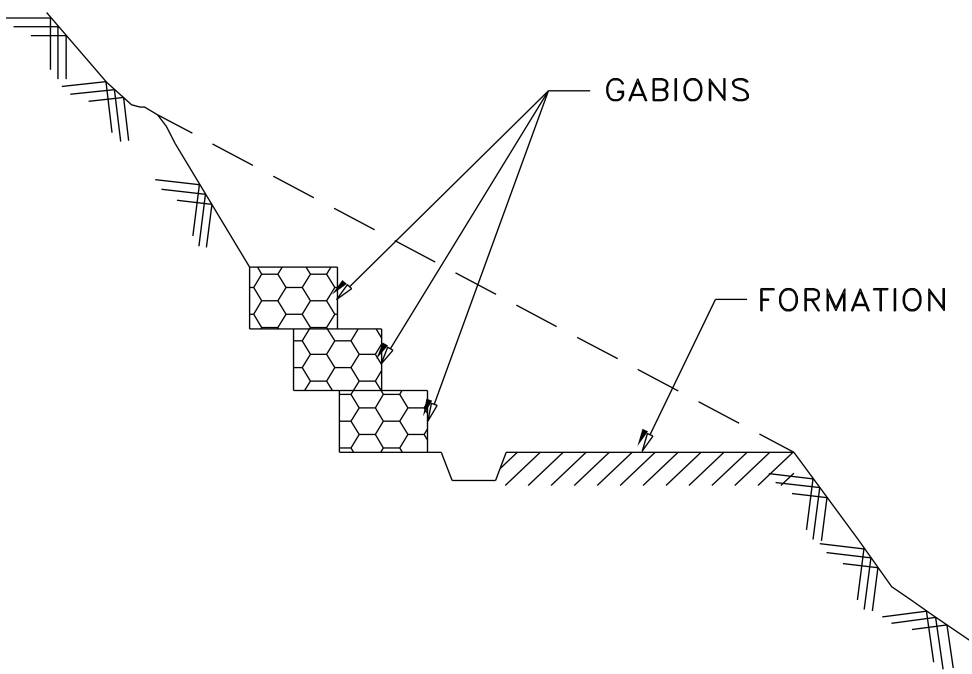


FIG. 9 : GABION WALL

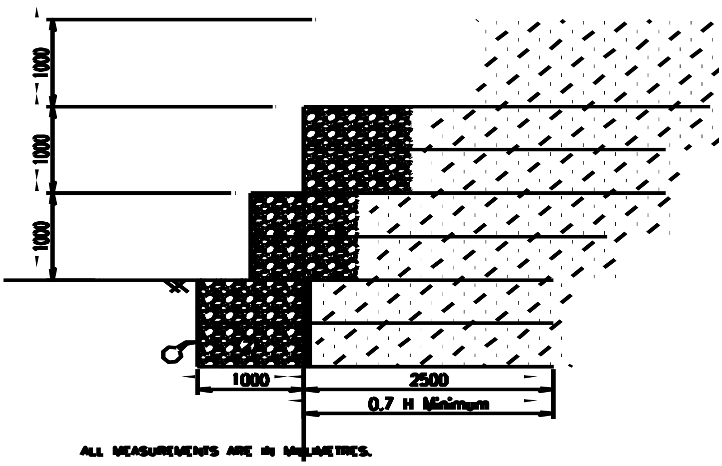


FIG. 10 : REINFORCED EARTH GABION WALL

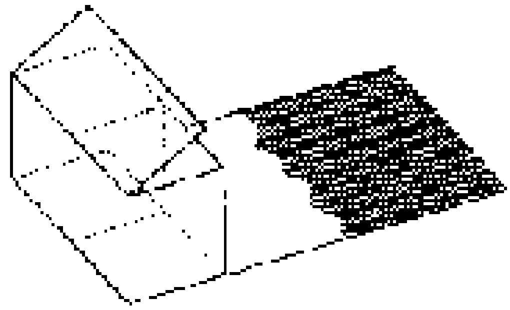


FIG. 11 : A WIRE NET CRATE FOR USE AS GABION

The biggest advantage of gabion wall support is its inherent flexibility. This enables it to deform or bend, rather than crack and collapse under alternating tension and compression. These gabions have high degree of permeability with no weep holes necessary. The gabions can be constructed in rows, preferably in a staggered manner. The damage to gabions is usually localised and requires only small scale repairs.

One drawback that has been observed occasionally is that falling boulders may cut or break the wire mesh, thereby leading to the possibility of stones falling out of the sausage crates. However, with adequate attention, such damages can be rectified and the integrity of the sausage walls maintained. If high humidity and other adverse climatic conditions prevail, rusting of wire may occur damaging the sausage casings.

<u>Advantages of gabions</u>

- Gabions are easy to assemble compared to construction of RCC/Mass Concrete/UCR Masonary wall.
- Provides more weight per unit cost and hence economical.
- Because of the open structure, they allow free drainage of water and relieve lot of pore water pressure and hence more stable.
- Unlike walls made of stone masonry, they have the advantage of being able to withstand large deformations without cracking and are flexible due to elastic nature of wire mesh and does not cause failure of retaining wall.
- Easy to maintain.

5.4.4 Providing/Placing of Geobags

These are like cement bags which are long lasting and less prone to wear and tear. These bags are filled with soil earth/sand and kept at the toe of disturbed/unstable soil slopes or in undermined locations. Geobags have been effectively used as a toe support in emergency, mainly as a temporary support measures. By systematically placing these bags and properly abuting with undermined soil slopes, further erosion and soil slip can be effetively controlled. However, these bags can be kept in place, in case no further disturbances are noticed at these locations. However, in highly disturbed and big soil slip locations, these need to replaced with toe walls of various types viz. Gabions, PCC/RCC walls etc. at a later stage for long term stability measure.

5.4.5 Rock Dowels

These comprise steel reinforcing bars that are cemented in to bore holes. These basically increase the shearing resistance across potential failure surfaces. A typical application is shown in figure alongside. They can also anchor restraining nets and cables, catch nets, catch fences, cable catch walls & cantilever rock sheds. The maximum available anchor forces is determined from the tensile and shear resistance of the steel cross section at the assumed failure surface crossed by the anchor.

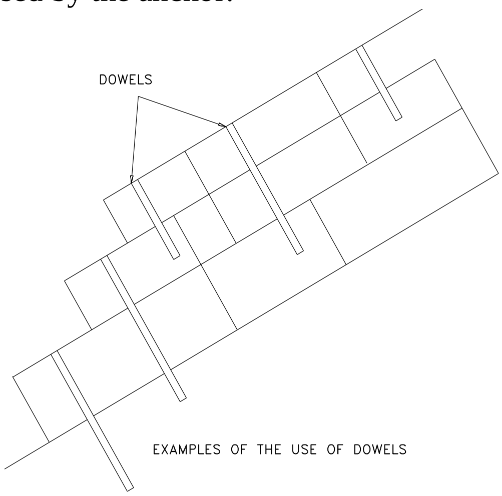


FIG. 12 : USE OF ROCK DOWELS

5.4.6 Rock Bolting

Rock bolts are used to strengthen, reinforce or tie together unstable blocks or beds of rocks. Rock bolts are low capacity reinforcement comprising a bar or tube fixed into the rock and tensioned to a predetermined load. Rockbolting is generally applied to jointed/ blocky rock strata to secure slabs and blocks on planes of separation that are inclined towards the excavation. These rock bolts are generally installed in a pattern & grouted with fast setting cement capsules. The use of rockbolts is normally confined to keep in place shallow layers of rock.

Rock bolts tie together blocks of rock to increase the effective base width in order to prevent toppling or to increase the resistance to sliding on discontinuities surfaces. It also anchors structures such as retaining walls and catch nets as shown in figure below.

For most rock bolts, the free length is fully bonded using grout or resin soon after the bolt has been stressed. If the surface layers of rock are in a weathered state, the bold head is likely to experience a loss in tension. Under such conditions, the rock face under each bolt head must be protected by fixing wire mesh set in shotcrete. Normally the design load does not exceed 60% of ultimate strength of bolts.

In the case of a rockbolt, the pre-compression is applied to the surrounding solid rock though tensioning of an anchored bolt. In pre-compressed rock, the critical factor is the strength of the anchorage. A rock which is to be strengthened by bolting, must therefore be sufficiently strong and continuous on a limited scale to provide a firm anchorage for the bolts, although under most conditions, anchorage may be provided be cement grouting.

It holds together the two strata of rock and prevents their mutual sliding. The anchoring becomes useful in following ways:

- The rock is reinforced by steel elements, which take over the tensile and to some extent shear stresses also.
- The rock medium is locked by the pre-stress bolts and this brings into effects the frictional forces along the natural plane of discontinuity.

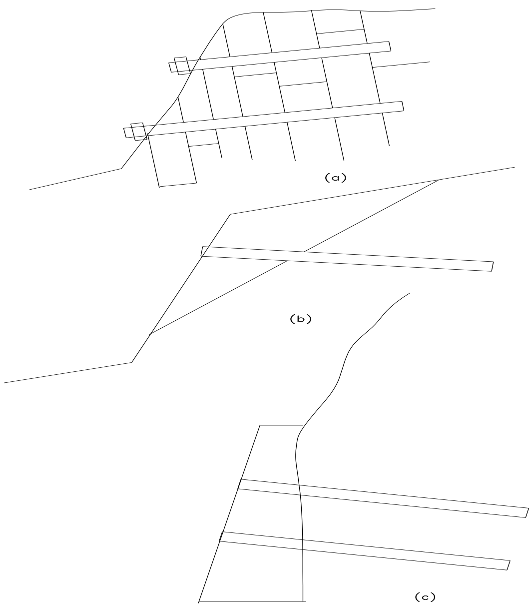


FIG. 13 : TYPICAL APPLICATION OF ROCK BOLTS AND ANCHORS

The degree of pre-compression available from the use of rockbolts depends on the diameter and strength of a bolt, the strength and deformation properties of the anchorage rock and the density of bolting. Of these, the second is critical in the determination of the other two. The effect of this precompression on rock properties will depend on its relationship to the principal stresses acting on the rock.

The maximum load (W) per bolt will then be given by W = a x $S_{c.}$ where a is the cross sectional area of the anchorage and S_c is the compressive stregth of the rock, and the precompression (σ_p) applied to the rock is given by

$$\sigma_p = nW/A$$

Where n is the number of bolts, and A the total area of bolted rock (assuming even distribution).

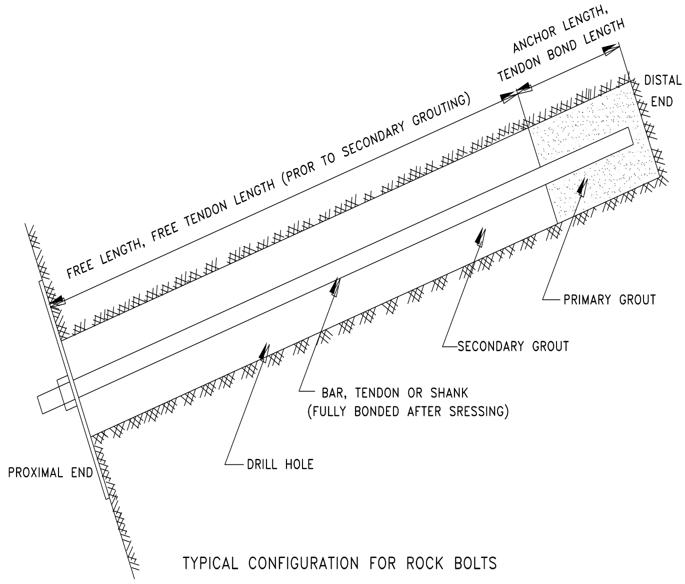


FIG. 14 : DETAILS OF ROCK BOLTS

5.4.7 <u>Rock Anchors</u>

Where unstable blocks of large size are involved or when it is required to stabilize a rock slope against the possibility of deep seated failure, the use of cable anchors, which are longer and of higher capacity than rock bolts, is preferred. They generally comprise tensioned cables which are fixed into rock. Cable anchors have also proved efficient as points of support for retaining walls.

Before grouting fixed end of anchor, it is necessary to grout fissures in the rock. The procedure therefore should be making the boreholes, water pressure test and if the boreholes stands water pressure, grouting the fixed end of anchors. Otherwise, grouting the borehole and reboring and grouting the fixed end of anchors thereafter. After the grout has set, reinforcing cable is stretched to impart a calculated magnitude of pre-stressing force to the rock mass and then the cable is fixed to the anchor plate. Pre-stressed anchor is an expensive remedial measure and is used only where there is no other alternative

5.4.8 <u>Micropiles</u>

Instead of retaining walls, piles are often constructed, because little space is needed for their instalment and thus the amount of excavation work is reduced. Also, sometimes slip circle may pass below the rail formation or below the toe of cutting in soil cuttings. This may cause upheaval of formation and shift/upliftment of the rail tracks. At such locations, retaining walls are not able to withstand the movement of soil mass behind and therefore they may fail. In such situation, piles/micro-piles are useful to stabilize entire hill mass behind them. These piles are driven below the critical slip surface and into the hard/in situ ground in order to bear the lateral/downward thrust entire disturbed soil and prevent its movement towards the track.

Micropiles are used at close spacing to increase bearing capacity and shear strength of sub-soil which is very weak. The diameter/depth of piles are decided after carrying out sub-soil exploration and on the basis of stability analysis. Micro-piles are cement grout/reinforced concrete (bored cast-in-situ) piles, generally 100 mm to 200 mm dia with central reinforcement as per design and driven in two to four rows at 1 m to 1.5 m spacing in a staggered manner, at least upto 4 m depth in firm/hard ground. Normally, steel casing pipes are used in loose soil/clay strata and upto beginning of rocky strata. Flushing with air and water is done before cement grouting.

The top of the piles are connected together with a pile cap to give fixity at the end. Over the pile cap, sometimes, gabions or concrete walls are erected to prevent the movement of loose surface soil earth from cutting slope, towards the direction of rail track in monsoon season.

5.4.9 <u>Timber Piles</u>

Timber piles are used for stabilization of small and relatively shallow potential slides. For stabilizing the slope, the piles, preferably of hard wood, are driven vertically into the slope with the help of hammer, operated manually or by any other suitable means. The size, spacing and number of rows of the piles may depend upon the local requirements. In a slide area, the piles may be driven into the body of the slide so that the same are anchored into firm strata beneath. Such groups of piles/rows can be repeated on top and toe in shallow failure areas of the slide, if necessary.

The piles should not be driven into soils which are susceptible to liquefaction. The longevity of the wooden pile may be increased by treating then with protective agents such as creosote oil. Such a treatment is essential if the area is infected with insects like white ants, etc.

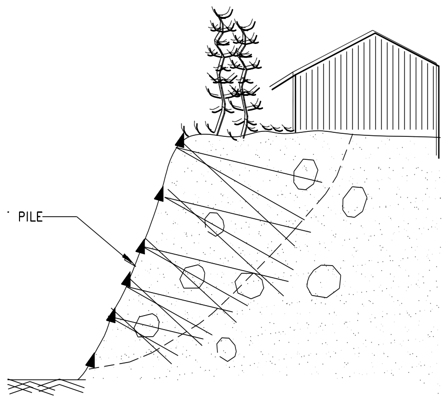


FIG. 15 : TIMBER PILING FOR SLOPE STABILIZATION

5.4.10 Soil Nailing

Wherever slope flattening is not possible, properly designed soil nails have been provided over the soil slope to increase overall stability of soil mass. Soil nailing is a soil support system in which in situ soil strata in the cutting slope is reinforced with the help of nails. The objective of the soil nailing is to create the reinforced soil mass that has sufficient internal stability so that it will provide additional safety factor against movement due to sliding.

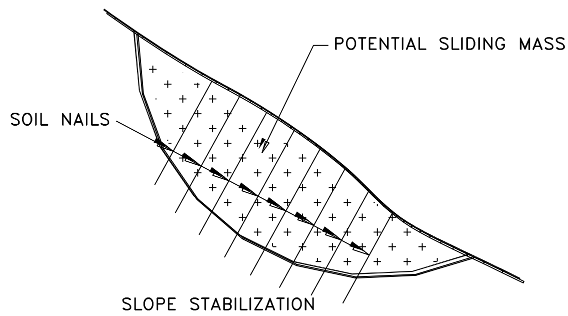


FIG. 16 : SOIL NAILING OF A FAILURE PRONE SLOPE

Soil nails are not tensioned. They are passive soil reinforcements. These nails usually consists of solid tor/steel bars of dia generally 20 mm to 50 mm, and length depends upon soil strength and its condition. The nails are normally placed at an inclination of 0 degree to 30 degree with the soil slope.

Once installed across a potential failure plane, tension developed in the nails provides resisting forces which stabilize the soil mass. The tension in the nail is developed by friction and adhesion at soil/nail interface. In addition to tension, resistance to bending also contributes to the reinforcing capacity through the nail acting in shear. Further, the face is protected by a chain link mesh.

For installing soil nails, the holes of appropriate diameter are drilled. Casing pipes are used, if necessary. Drilling rig is used to excavate soil and subsequently nails are pushed into the holes. The drilling casing is removed from around the nail and the annular space between the nail and borehole is grouted with cement or suitable material. For permanent applications, nails are epoxy coated for preventing corrosion. These types of nails are known as ‘Grouted Soil Nails’

5.4.11 Crib Walls

Crib walls can be used as restraining structures and may be made of reinforced concrete or steel members. The vertical posts are connected by horizontal members and allow free drainage. These are suitable for small slides that are not deep seated. An arrangement is illustrated in figure below.

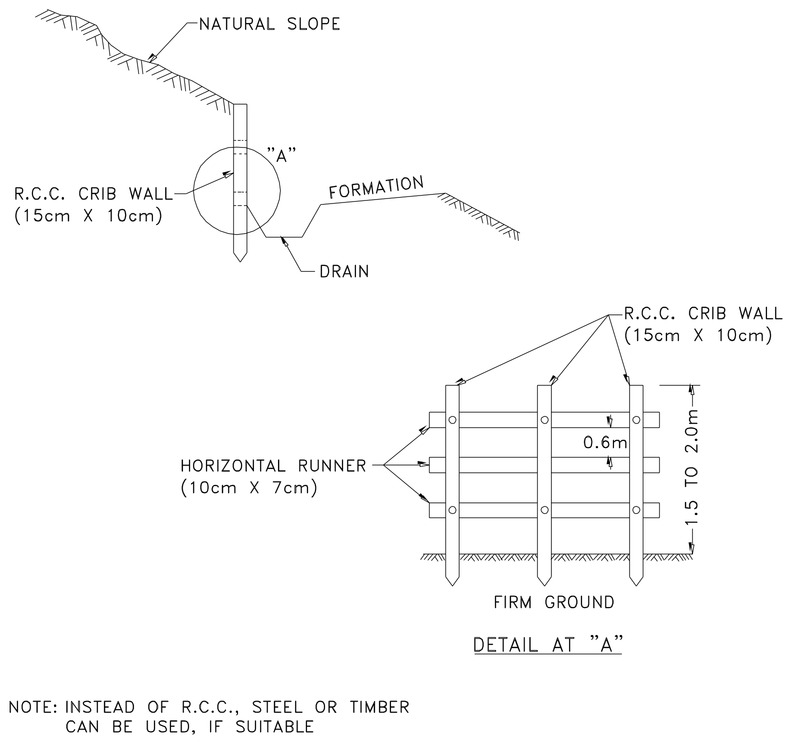


FIG. 17 : R.C.C. CRIB WALL

5.4.12 Bally Benching

Bally benching is used for control of surface erosion on slide areas as well as for arresting shallow movement of the top mantle of slide mass. This technique can also be used effectively in preventing the deepening of gullies/chutes, caused by the eroding action of flowing water. During rains, the surface flow generally results in gully formation on slopes. Such gullies, if allowed to deepen, induce instability in the slide slopes which eventually fall/slide down. Barrier/bench system helps densification of soil material surrounding the ballies, thereby increases the strength of the slope, and prevents the shallow movement of the loose mantle and retards the speed of surface water responsible for gully formation. Typical arrangement consists of ballies driven vertically and tied horizontally in rows parallel to the track along the hill slopes above and below the formation level as shown is shown in the figure below-

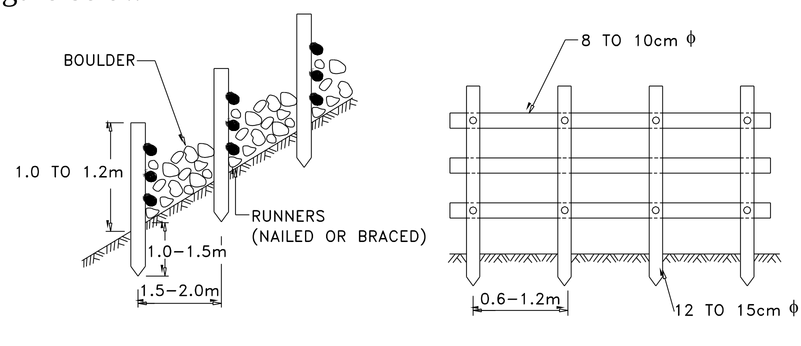


FIG. 18: BALLY BENCHING

5.4.13 Stabilization by Planting

The aspect is generally ignored, though it is very important. It involves promotion of vegetation with or without the help of mulching, jute netting, coir netting or geotextiles. It is economical and fruitful if local species are specially nurtured to grow on a particular slope or the area.

Reforestation of a slope disturbed by sliding is an effective controlling measure only of shallow debris slides. Landslides with a deep-lying slide surface cannot be stopped by development of vegetation, although this does reduce infiltration of surface water. The forest growth is generally believed to assist in drying out the surface layers and in consolidation of the slope by the ramifying root system. The most suitable species of vegetation are those that have the largest consumption of water and the highest transpiration rates. Reforestation is carried out in the later stages of remedial work, after at least some degree of stabilization has been achieved.

5.4.14 Use of geosynthetics

Earth retaining structures with vertical faces can be built using concept of reinforced earth. The other alternative in such a situation is construction of high retaining structures or viaducts which are costlier. Initially, horizontal metal strips of galvanized mild steel, aluminium and stainless steel were used as reinforcements. With the advent of geosynthetics having high tensile strength, these are now used as reinforcement in the reinforced earth. Internal as well as external stability of reinforced earth fill must be ensured by proper analysis and design.

Reinforced earth fills require granular material since good frictional resistance is required for developing the reinforcing effect. The general reinforced earth has good deformability characteristics and as such, its behaviour is satisfactory, even under conditions where the hill slopes experienced movements. In such situations, rigid structures would suffer damages.

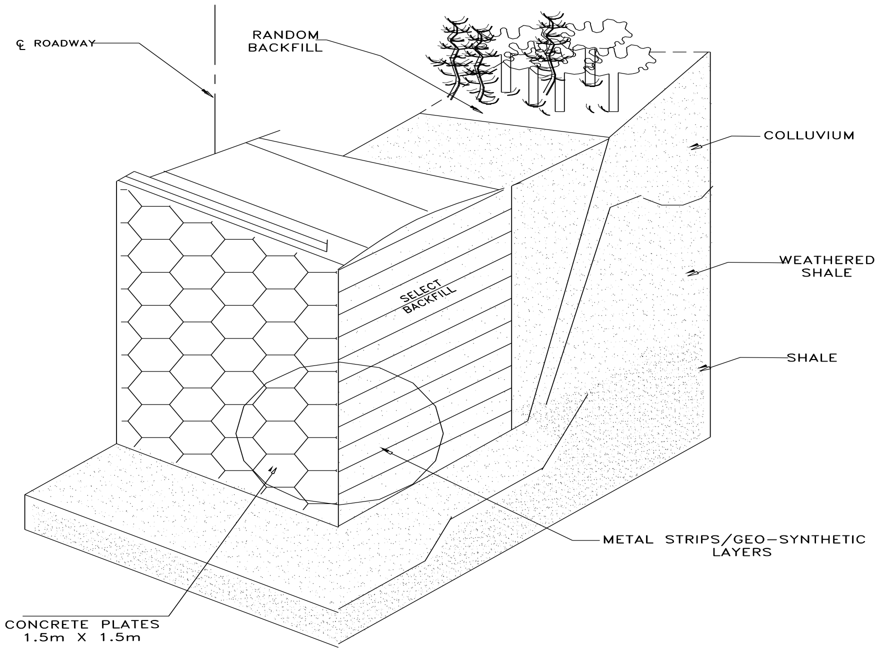


FIG. 19 : USE OF GEOSYNTHETICS IN SLOPE STABILIZATION IN CUTTING.

5.5 Damage Control and Impact Reduction Systems

5.5.1 Anchored Cable Nets

They can be used to retrain masses of small loose rocks or individual rocks that protrude from a rock face. In principle, an anchored cable net performs like a sling or reinforcement net, which extends around a surface of the unstable broken rock to be supported. The cable net strands are gathered on each side by main cables leading to rock anchors.

5.5.2 Anchored Wire Mesh

Wire mesh is versatile and economical material in providing protection against rock fall where small blocks (blocks smaller than 0.6 to 1.0m size) are involved. Layers of mesh are pinned on to the rock surface to prevent small loose blocks of rock becoming dislodged. Mesh can also be used essentially as a blanket draped over the rock surface to guide falling rock in to the ditch at the base of the slope. The same arrangement can also be used on stony overburden slopes to prevent dislodged stones from rolling down the slope. Mesh can be combined with rock bolts to provide generally deeper reinforcement. It should be used if the slope is uniform enough for it to be in almost continuous contact with the face.

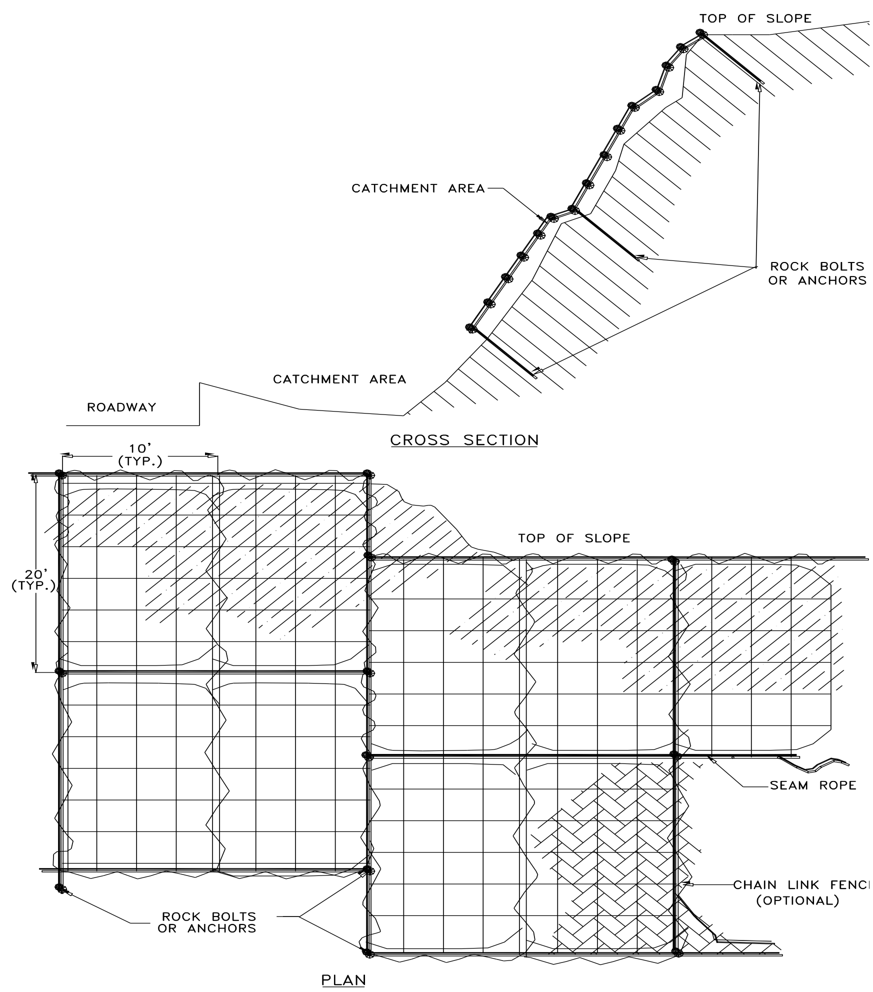


FIG. 20 : STEEL WIREMESH PROTECTION ON CUT SLOPE

This solution is effective in the following situations-

- Where, the rock strata is fairly strong, but jointed & blocky in nature, with no definite pattern/ orientation of joints.
- Where rock mass is shattered, with wide/deep & uneven cracks which can not be removed by loose scaling.
- Where big size boulders are seen in top rock slopes, partially embedded in soil.

5.5.3 **Bouldernets of Geosynthetics**

Rock fall hazards can be mitigated by use of *geosynthetics.* These are manufactured from selected polymers by a special process that aligns the molecular chains of polymers and thereby producing material of high tensile strength and high resistance to natural ambient condition. In this protection technique, the susceptible slope is covered with geosynthetic mesh. To fix the geosynthetic on the slope, stable part of the slope area not forming a source of rock fall has to be first identified and located. This area will normally be situated some distance beyond the periphery of the slope affected by instability. At their end, suitable anchors rigidly fix the geosynthetic mesh. The geosynthetic roll is fixed to the iron bolts with the help of clamps and nuts. The geosynthetic roll is then rolled down the slope ensuring that the roll generally follows the contours of the slope.

5.5.4 <u>Catch Ditches</u>

Catch ditches serve as important tool in containing rockfalls. They have been covered in para 2.0 of Chapter IV.

5.5.5 <u>Catch Nets, Catch Fences & Catch Walls</u>

Wire mesh can be effective in intercepting or effectively slowing bouncing rocks as large as 0.6 to 1.0 m when the mesh is mounted as a flexible catch net rather than as a standard, fixed wire fence. If suspended on a cable, the mesh will absorb the energy of flying rocks with a minimum of damage to the wire catch net as shown in the figure below.

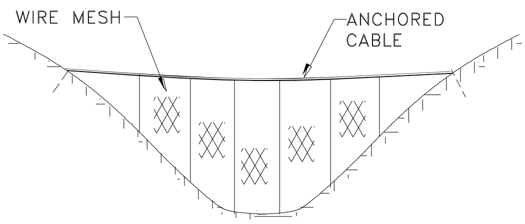
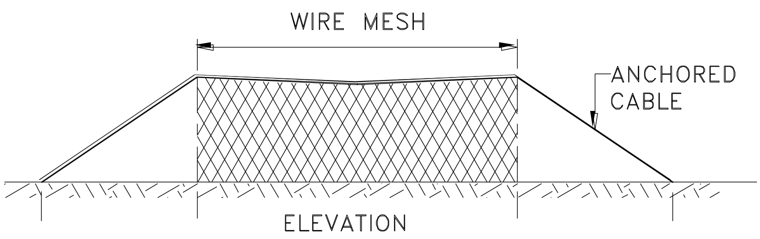


FIG. 21 :   CATCH NET OVER GULLY

The principle of the catch fence is similar to that of a catch net. Its purpose is to form a flexible barrier to dissipate the energy of rapidly moving rocks. The fence should be suitably situated so that accumulated rocks can be removed easily. Various arrangements of catch fence are shown in figure below**.**



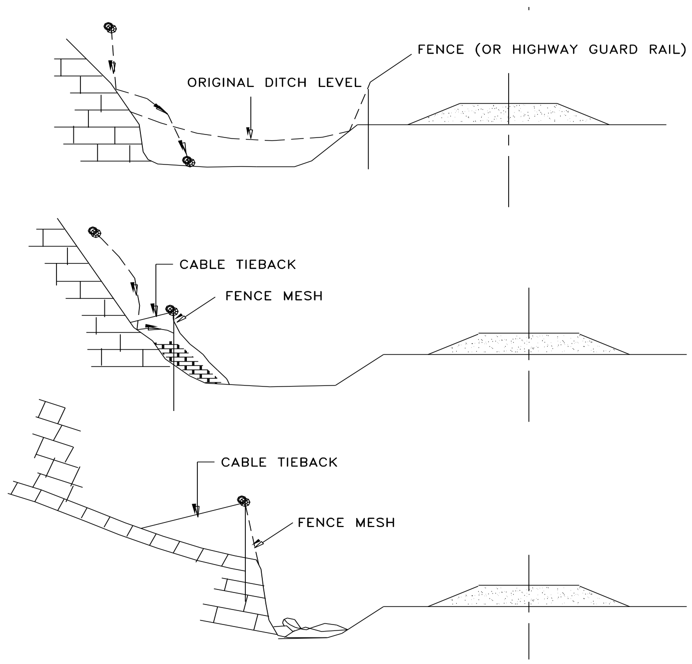


FIG. 22 : CATCH FENCES ACROSS SLOPE

Catch walls can be used to form a barrier to stop rolling or bouncing rocks as large as 1.5 to 2 m from reaching the track. They usually increase the storage capacity of the ditch so that maintenance intervals can be extended. In many locations, lage catch ditches may not be effective in intercepting large rolling rocks without use of catch walls. To achieve maximum protection and storage capacity, the catch walls should be located on the side of the ditch closest to the track. Catch walls can be constructed using reinforced concrete or gabions**.**

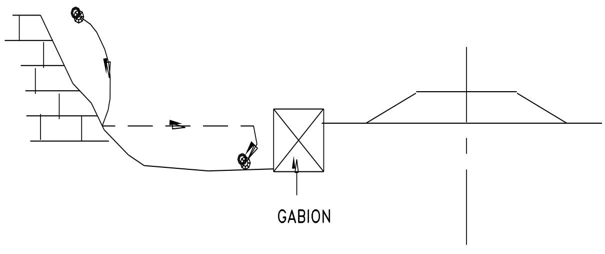


FIG. 23 : SHAPED DITCHES WITH GABIONS

The gabion catch wall is a flexible structure that, upon setting or being hit by impact, tends to deflect and deform instead of break. Gabions therefore make good inexpensive catch wall structures.

5.5.6 <u>Rock sheds & Tunnels</u>

When other forms of stabilization and protection methods are ineffective, rock sheds & tunnels can be used for protection against rock falls and slides depending upon the site conditions. This option should be considered in areas with serious problems as it gives complete protection. This method of protection is expensive and maintenance cost is negligible. Experience is required to decide on the most suitable type of structure and the loads to be carried. A rock shed should be able to resist the energy transmitted by the largest rock mass likely to pass over it during its life, hence probability analysis should be involved. Transmission of energy in falling process depends on type of movement of rock mass as free falling, bouncing or rolling.

CHAPTER VII

LATERITE CUTTINGS – CONSTRUCTION AND MAINTENANCE

1.0 <u>SCOPE</u>

Planning, construction, maintenance and rehabilitation of cuttings in lateritic formations form the scope of this Manual. The procedures detailed in this Manual are based on established practices in Railway cuttings in coastal areas, hill road construction in various states, published literature and investigations on landslides.

2.0 <u>GENERAL FEATURES OF CUTTINGS</u>

2.1 Earthwork in forming bank and cuttings is the most common and voluminous activity in rail road projects. An extra care taken in understanding geology of the natural ground and assessing its geo-technical properties in all weather conditions will pave the way for ensuring long term stability with the least maintenance attention.

2.2 Earthwork in cutting the natural ground, greatly reduces the confinement of in-situ soil hidden cleavages and cavities in the strata get opened up and natural drainage pattern gets altered including draining out of wells. As the strength gets reduced due to unconfinement of face of cutting, cuttings become vulnerable for slope failures. As per theory, factor of safety decreases over a period of time for cuttings whereas it increases over a period of time for banks. **<u>Drainage is the most important factor contributing for health of cuttings.</u>**

2.3 The salient features of the cuttings are:

(i) Ground level, formation level and depth of cutting
(ii) Formation width, width of side drains and formation treatment required, if any.
(iii) Slope of cutting based on strata properties.
(iv) Catch water drain, bottom side drain and drains at intermediate depths, longitudinal slope, width and depth of drain, functioning of drain.
(v) Berms to break the single slope so as to localize the slips; berm width from geo-technical and functional requirement enabling access, compaction, repairs, etc.; protection of berm by paving and to prevent water ingress; corner protection to prevent erosion.
(vi) Land width at top, land use at top, service road facility, general terrain slope away from cutting or towards cutting.
(vii) Protective works -

- Earth retaining wall with weep holes, size, inclination and spacing of weep holes, functioning of weep holes, graded back fill; rammed puddle clay at bottom layer of back fill; construction joints; stability against sliding and overturning, bearing capacity, settlement.

- Pitching – dry or mortared, thickness and weight of stones, backfill.
- Gabions, reinforced earth gabions.
- Protective vegetation and plantation.

3. LATERITIC FORMATION

3.1 Lateritic formation exists world over mainly in tropical countries in heavy rainfall areas. It is a product of tropical or subtropical weathering in a monsoon climate having alternate dry and wet seasons.

3.2 Laterite is broadly defined as a product of intense rock weathering, generally the crust reddish in colour having a vesicular and vermicular appearance, soft when freshly quarried but hardening on exposure. Laterite may form from variety of rocks, the end product containing mainly hydroxides of iron, alumina and manganese. The original alkalis, lime, magnesia and silica are removed in solution, silica being present until late stage. In accordance with the relative amounts of ferric oxide, alumina and manganese present, the material is called ferruginous laterite, aluminous laterite and manganiferous laterite, differing in colour.

3.3 Material formed in-situ is called primary laterite and when this material is transported to other places and there recemented, it is called detrital laterite. Primary laterite material is compact and fairly uniform in composition whereas the detrital laterite is heterogenous and far from compact. Classification of laterites into high level and low level (coastal) laterites also exists. The term `laterite' was suggested by F.H.Buchanan (1800-1807), an Officer of East India Company and the word derived from Latin word `later' meaning brick.

3.4 The top crust is usually quarried in the form of bricks for construction purpose. This hard crust is a mixture of oxides of iron and alumina and iron oxides, generally, preponderate and give the crust its prevailing red colour. Between hard crust of ferruginous laterite at top and the parent rock at bottom, the layers of gravelly moorum, silt and clay are found. These soils are basically weathered products of parent rock and differ in colours. Moorum is extensively used in banking and has excellent engineering properties.

3.5 A soft horizon of clayey weathered formation dominated by Kaolinite is termed as `Lithomargic clay', derived from Greek word `lithos' meaning rock and the Latin word `marga' meaning heavy clayey earth. This is used for pottery, chalk making, decorative Rangoli and medicinal purpose but has poor engineering properties.

3.6 Between lithomargic clay and parent rock, soft rock may or may not be present and this is not a competent stratum as it is in a geological process of weathering.

3.7 Decomposition of primary minerals, laterisation by leaching of combined silica and bases and dehydration of colloidal hydrated iron oxides are the three stages in the process of laterisation. Geotechnical properties are primarily influenced by degree of physico chemical weathering.

4.0 SPECIAL FEATURES OF LATERITIC FORMATION

4.1 Generally, it is expected that deeper the excavation, harder the ground met with but in lateritic formation, this trend is reversed. Hard stratum is met at top followed by soft stratum of weathering zone for considerable depth and the parent rock at the deepest level. Geo-technical requirements are more demanding as we go deeper due to geology of formation. Strength decreases as one goes deeper due to varying engineering properties.

4.2 Engineering properties of soil do vary during summer and monsoon periods and hence inferring results of bore log samples taken in summer may not suffice for taking major engineering decision unless supplemented with sampling during monsoon. High SPT values may be recorded when the sampler strikes against hard pieces of laterite which may lead to wrong conclusion. Continued sampling and sampling under wet conditions only can reveal the true properties to take major engineering decisions.

4.3 Hard and soft portions of laterite are to be dealt with diligently. More requirements of explosives, increased ground vibrations, reduced output and reduced fragmentation are characteristics of porous hard portion of laterite crust at top. The losing of strength and behaving like a liquid on coming into contact with water is the characteristic behaviour of soft portion of laterite, called lithomargic clay. It has good shear strength in dry condition but negligible shear strength in saturated condition. Slope failures due to natural shear zones in the strata and land slides due to pore pressure build up are common in many tropically weathered laterite soils during rainy season.

4.4 Laterisation is a continuing process and soft portions become harder gradually in monsoon and tropical conditions. The reddish brown colour on side surface of cutting after excavation may be deceptive to be hard as the strata behind may be soft and is in the process of laterisation. Local slips may occur consequent to insufficient hard cover on sides of cutting.

4.5 Laterites are pervious and store a good amount of water. Springs are seen commonly oozing from the flanks of laterite plateau at the junction between porous laterite and the underlying impermeable or unweathered strata. Sudden gushing out of water during excavation or sudden draw down during excavation due to natural cavities below are quite common. Ground subsidences are common in intense rainfall condition and in surcharged ground condition.

4.6 The detrital laterite or reworked laterite is formed in later ages geologically and this is geologically unstable. The rolling down of unstable laterite into valley portion and forming tunnels and caverns is a characteristic of this formation. Sudden sinkages of earthwork machines during excavation due to natural cavities are quite common. Formation continues to move in the geological process to attain stability which may endanger safety of structures founded / located on reworked lateritic zone.

4.7 On hard crust at top as well as in lithomargic zone, seldom plants grow and hence plantation to bind the soil to avoid slips is not feasible.

4.8 Strata is generally heterogeneous and arch action for supporting the loads is not reliable. Greatest influence on slope stability is the variability of soil properties. Heterogeneity of tropically weathered laterite soils in cuttings makes realistic analysis on slope stability difficult. Stability predictions are further complicated by such factors as the degree of weathering, surface erosion and the hardening or softening of some newly exposed cuts.

5.0 PLANNING AND DETAILING OF CUTTINGS

5.1 The hard crust at top generally requires blasting or heavy-duty excavators for excavation. Planning the side slopes of cutting based on this top hard strata is not a correct practice and decision based on borehole data is a correct practice.

5.2 Planning land acquisition based on hard crust at top is also not a correct practice as soft portions may occur at deeper levels and widening from top after reaching particular depth is a difficult process. Land acquisition for widening of cutting due to late realization of meeting soft soil is a time consuming activity.

5.3 As excavation efforts vary along depth of cutting, quantity estimation in various strata shall be based on bore logs. Classification of laterite requiring blasting and not requiring blasting as well as laterite requiring controlled / intensive / normal blasting shall be objectively done with bore-logs and proximity to sensitive structures rather than subjectively assessing.

5.4 As far as possible, formation level fixing in lithomargic zone may be avoided. To balance the quantities in banking and cutting, deeper cuttings may result in but is preferable to avoid deep cuttings with formation level in lithomargic zone.

5.5 It is advisable to break the monotony of slope of cutting along depth and along length. Along depth, berm can be provided at every 6 m depth or wherever there is an abrupt change in type of soil. Berms at intermediate depths will tend to avoid slips for entire depth but it will be localized to slips between the berms. To localize the slips between the berms, berm width shall be adequate, say shall be 3 m. A berm of 3.5 m width, will help accessibility, transportation, compaction and drainage (side drains). To minimize the length of damages due to slips etc., it is preferable to break the slope along the length of cutting by providing more trolley refuges so that slips will be localized and restoration time will be minimal after slips.

5.6 Slips at later years cannot be ruled out in lateritic strata. Access problem is acute in slip restoration, especially removal of slipped earth which is wet. Handling moist earth is a difficult work and normally it is allowed to dry and then transported. For this additional trolley refuges more than what is stipulated in SOD are preferable.

5.7 Pattern of land slides indicate deep slips in case retaining wall is rigid and pore pressure build up due to absence of drainage through weep holes, mostly choked / clogged. Weep holes initially provided may become not functional over years due to lack of maintenance. A flexible retaining wall with construction gap at every 6 m, will allow yielding in case heavy slipping energy to be dissipated. Reinforced earth gabion walls are preferable to localize the slips as well as converting deep slips to shallow slips. If these are proposed to be provided, then deformations have to be checked with calculations so as not to infringe moving dimensions.

5.8 A good drainage speaks health of cutting. Catch water drain at top, side drain at bottom and drains at intermediate depth say at berm level, will improve stability many fold. Sides of drain shall be paved with corner protection with chicken mesh so that water will not ingress into soil and bypass the drains. Provision of drainage through retaining wall will enhance the stability of retaining wall as well as cutting. Conventional weep hole system is not effective in long run. A dry rubble wall with mortared bands is an appropriate solution and this is an established practice in hill road construction. This is a time tested practice of release of pore pressure build up. In lieu of dry rubble, heavy weight laterite stones or precast concrete blocks with rough finishing can be used. Typical arrangement combining Paras 5.5 and 5.8 is shown in Fig.1.

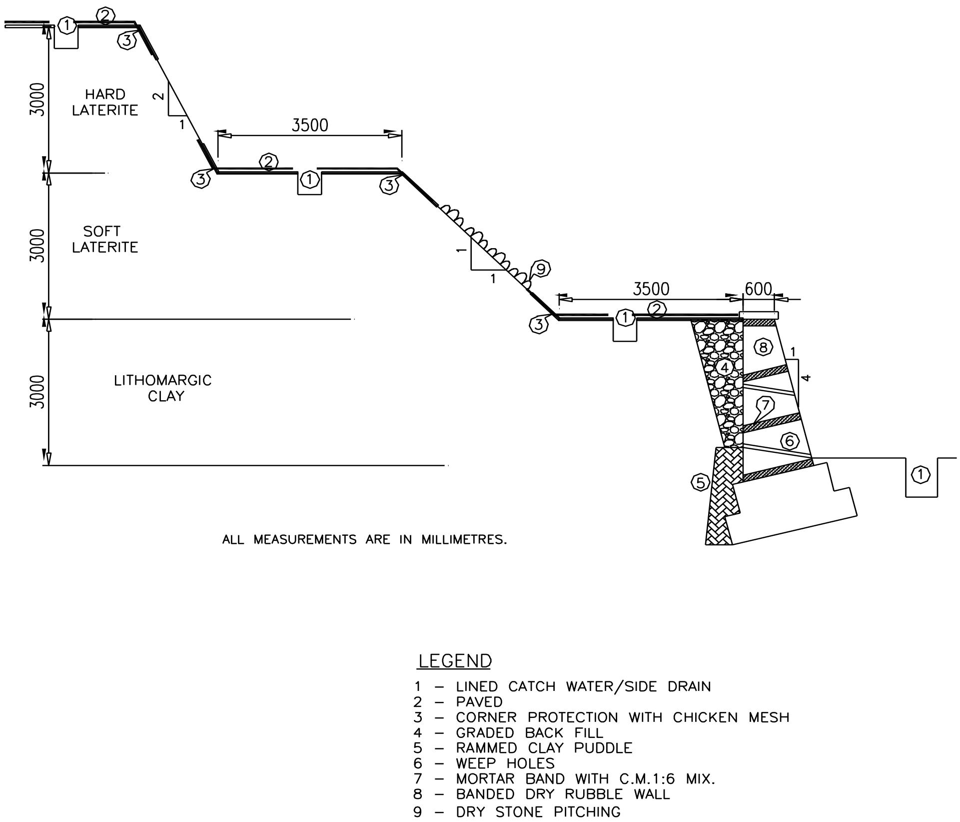


FIG.1: SLOPE PROTECTION WORKS FOR LATERITE CUTTING

5.9 In a strata with laterite boulders, removal of boulders may cause more instability. Blasting will cause further instability. Catch pits at formation level with minimum 2 m width on either side, will enable boulders to fall in catch pit rather than infringe the track and hence restoration will not be a problem. Incidentally this will accommodate slipped earth with boulders.

5.10 Side long cutting is more vulnerable for slip damages than both sides cutting. Berms at 3 m depths and flatter side slopes of 1:1 are preferable for long term stability of cutting. Single slope shall never be attempted for soft soil cuttings and cuttings in bouldery strata for depths more than 3 m.

5.11 All the reddish colour soil does not conform to laterite soil. Reddish colour soil with more of silt and practically no cohesion may also give a deceptive look of laterite soil. Another variety with hard crust on outer periphery and hollow in the core also gives a deceptive look of laterite, popularly called in local language `NARIKAL' meaning FOX stone. SPT values will be high because sampler hits the hard crust but in reality no strength and sudden shear failures are not uncommon. A thorough investigation is needed to qualify a soil as lateritic soil.

5.12 Blanketing has to be planned if the strata at formation level is soft with poor bearing capacity in moist condition.

5.13 Bridges are designed for floods with 100 year return period for foundation design and 50 year return period for superstructure and waterway. Similarly cuttings also shall be designed for 100 year rainfall and intensity of rainfall measured in mm/hour is more important than magnitude of rainfall in mm. Rainfall from above the cutting and water table raising from below the cutting and associated drainage pattern have to be adequately modeled for predicting land slides. Excessive precipitation is considered to be the main triggering agent of the land slides occurring in Western Ghats. Landslides and ground subsidence are probable if Southwest monsoon is immediately followed by Northeast monsoon resulting in surcharged ground conditions.

5.14 Cost economics of cut and cover versus deep cutting in soft soil may have to be worked out and decision then taken taking into account maintenance efforts required. Generally for 30 m deep cutting in lateritic formation with lithomargic clay at bottom, cut and cover is cheaper and more durable than deep open cutting with protective works. Land width requirement is less and land use is efficient in case of cut and cover. Cutting over cover shall not be left unprotected and drainage over cover in cutting is very important. Safety of personnel while working in lithomargic zone is of primary concern as many cases of workers buried alive due to caving in have occurred.

5.15 In case of cutting in soft soil more than 30 m depth, open cutting with protective works as well as cut and cover may not be appropriate. Either tunnelling or diaphragm wall with ground anchors is to be proposed as per cost and time economics. Face collapses do occur in tunnelling in lithomargic zone depending on standup time and hence buttressing the face has to be planned with or without shield.

5.16 While proposing tunnel / cut and cover / open cutting adjoining running track and tunnel, safety aspects have to be specially planned and side soil of existing tunnel shall not cave in during execution.

6.0 CONSTRUCTION TECHNOLOGY

6.1 The hard crust at top, usually requires to be blasted for excavation. As the strata is pervious, blasting requires more energy input compared to granite and outturn will be less. Blasting opens up cavities and joints inherent in lateritic strata and thus affects long-term stability. As lateritic formation contains water bearing strata / aquifers, vibration level will be more when the blasting release energy waves pass through such strata. To take care of all these problems, use of slurry explosives is the appropriate solution.

Slurry explosives compared to NG based explosives have lower velocity of detonation which reduces and restricts induced structural damages to rock mass and surrounding masses. This is essential for lateritic strata with discontinuities, cleavages and cavities. They provide greater safety in handling, transporting and storing explosives. Cost of blasting is 25% cheaper.

In open cutting in hard laterite strata, experimental blasting was made with slurry explosives and with delay detonators for various drilling patterns and results are as follows:

S. No.	Spacing of 100 mm dia holes in `m’	Depth of hole in `m’	Charge factor kg/cum	Fly rock	Ground vibration	Fragmentation
1.	2 X 2	4	0.5	Nil	More	Frag size $\leq$ 0.3 cum
2.	2 X 2.5	4	0.4	Nil	Less	Frag size $\leq$ 0.4 cum
3.	2 X 3	4	0.3	Nil	Very less	5% oversize 1-1.5 cum rest within shovel size

Third pattern is proved to be optimal. Parallel and inclined drilling pattern with `sinking cut pattern’ improves fragmentation and reduces ground vibration. Similarly short relay detonators with 25 milli sec delay interval reduces ground vibration and improves fragmentation compared to half sec delay interval.

6.2 It is difficult to tackle huge size boulders of isolated type with conventional blasting or with excavators. Plaster shooting technique using detonating chord alone for blasting will be a preferable solution.

Plaster shooting is the most suitable method of breaking huge size boulders without drilling. Detonating cord alone (10gm/m) is wound round surface of boulder and knot is mud-capped. The charge required is 500 – 700 gm/cum.

Alternatively explosive charge is placed on the boulders preferably in a cavity or in between the boulder and ground and blasted after suitably mud-capping the charge. The disadvantages with this method are excessive noise and air blast.

6.3 If the excavations are done during summer and lithomargic clay and soft layers are protected during monsoon (not exposed to rainfall), slips during monsoon can be minimized. Protective works shall be completed in advance of monsoon, else, soft layers shall not be attempted to be excavated just before monsoon and left exposed during monsoon.

6.4 Generally, construction of retaining wall requires excavation to be completed for entire depth of cutting in advance. Time period for excavation and for construction of retaining wall has to be judiciously planned so that soft portions will not get exposed during monsoon. Any time lapse may have serious consequences leading to landslides which are difficult to tackle once occurred. A more efficient way will be to reinforce the soil with soil nailing, simultaneously with the excavation in layers, rather than wait for excavation for full depth to be completed so as to start construction of retaining wall.

6.5 Soil shall have high friction for the soil nailing to be effective. If the natural soil does not have adequate friction, inclined bore hole can be made, sand can be filled inside bore hole and nail shall be driven through this sand so as to ensure friction. Staggered spacing of soil nails will be preferable and 32 mm dia ribbed rods as soil nails will suffice. Use of special perforated tubes enables cement grouting by gravity or under low pressure. Soil nail shall have adequate anchorage length beyond potential slip plane.

6.6 For long length cuttings, drainage during construction stage is very important as cutting may not be completed in one season and intervening monsoon period may pose stability problems. This is also the case with deep cuttings. Temporary catch water drains at top or bunding at top, temporary longitudinal and cross drains at excavated level or pumping out shall be planned to ensure stability during monsoon and also to enable excavation after monsoon as it is dangerous to excavate saturated earth in post monsoon. It takes time for water to get drained in clayey strata and excavation immediately after cessation of rains is not feasible.

6.7 A practical approach to protect the lithomargic clay at bottom layers from vagaries of monsoon will be to excavate for more width during summer in soft zone, form compacted bank in summer in soft zone depth, leave it during monsoon and excavate the formed bank to the required profile of cutting after monsoon. This has two advantages. Soft portions of excavations are well protected during monsoon with formed bank and soft portions get adequate side cover of good earth after re-excavating to profile in post monsoon period. In fact, earthwork is cheaper than any of the established protective works / retaining works.

6.8 Retaining wall of gravity type requires adequate width and depth of foundation. Open excavation in soft layers may be hazardous some time and may lead to fatal accidents. Excavations shall never be for long lengths and preferably restricted to 6 m at a stretch from safety point of view. It cannot always be guaranteed that excavation during summer will always be safe as the strata may be water bearing strata containing aquifers or underground streams are possible or strata may be a buried river course, well or tank. Hence slips have to be localized during construction stage by reducing excavation length. If the excavation is hazardous, small dia hand augered piles shall be started at a higher level, augured to a required depth of foundation, pile cap and retaining wall constructed over it so as to completely avoid open excavation in soft portions. Fatal accidents can thus be avoided.

6.9 Protective works generally adopted are as follows:

(i) Mortared masonry or concrete retaining wall.
(ii) Banded masonry dry rubble retaining wall.
(iii) Slope protection with pre-stressed ground anchors or soil nailing.
(iv) Gabion wall with or without earth reinforcement.

Pre-stressed ground anchors require competent strata for anchoring at inclination and this may not be available within the acquired width. Land availability and competency of strata (soil anchor or rock anchor) have to be checked in advance.

Out of various possible combinations of protective arrangements, more appropriate would be to propose banded masonry dry rubble wall in bottom portion of cutting for 3m – 6m depth and dry stone pitching in upper layers with berms at intermediate depths with side drains at berm level. In hard laterite, unaffected even during monsoon, slope of 0.5 horizontal to 1 vertical may be proposed, otherwise slope of 1 horizontal to 1 vertical is preferable for long term stability. This scheme is covered in detail separately.

6.10 Pore pressure release through drainages, permeable backfill behind retaining wall, dry rubble retaining wall and dry stone pitching shall form part of detailing of cutting while execution. Counterfort drains and/or relief wells may supplement drainage to enhance stability of cutting. In addition to catch water drain, if a bund is provided on cutting side of catch water drain, overflow can be controlled in case of heavy rainfall falling onto cutting sides and eroding the soil. Rain cuts can also be prevented.

6.11 Excavated cut spoils as laterite spalls can be used for pitching of slopes, paving of drains on berms and for retaining walls. By choosing the explosive quantity, desired fragmentation can be achieved by trial and error.

6.12 To protect the sensitive structures from vibration of blasting, pre-splitting technique shall be used. Prior cracking of strata with delay detonators and controlled blasting to the required depth can minimize the blasting waves reaching the sensitive structures especially in water bearing strata.

6.13 Conventionally it is not advisable to dump the cut spoil onto top of cutting to prevent slips caused due to overburden. In lateritic strata with hard crust at top, this over burden may not pose problem but nevertheless, it is not a good practice to dump the cut spoil on top.

6.14 Cost economics of cutting with flatter slopes than cutting with steep slopes and with protective works may have to be worked out. Natural dipping planes in strata can be identified by inspecting the terrains, cut slopes and from bores. In case of dipping planes in longitudinal direction, i.e. sloping along length of cutting, elaborate protective arrangements are not required. In case of dipping along cross section, one side face only required to be protected which is unfavourably sloping down towards the track. Under intense rainfall condition, in many cases of landslips, cuttings with flat slopes and without retaining walls had survived whereas cuttings with steep slopes and retaining walls had slipped due to retaining walls acted as barriers of free drainage.

7.0 RETAINING WALL

7.1 Dry rubble wall with horizontal and vertical mortared bands is commonly adopted in hill road construction in various states. This arrangement has significant advantage of permitting drainage through the wall cross section and thereby release of pore pressure build up which is the basic requirement of cutting. Typical arrangement is shown in Fig.2.

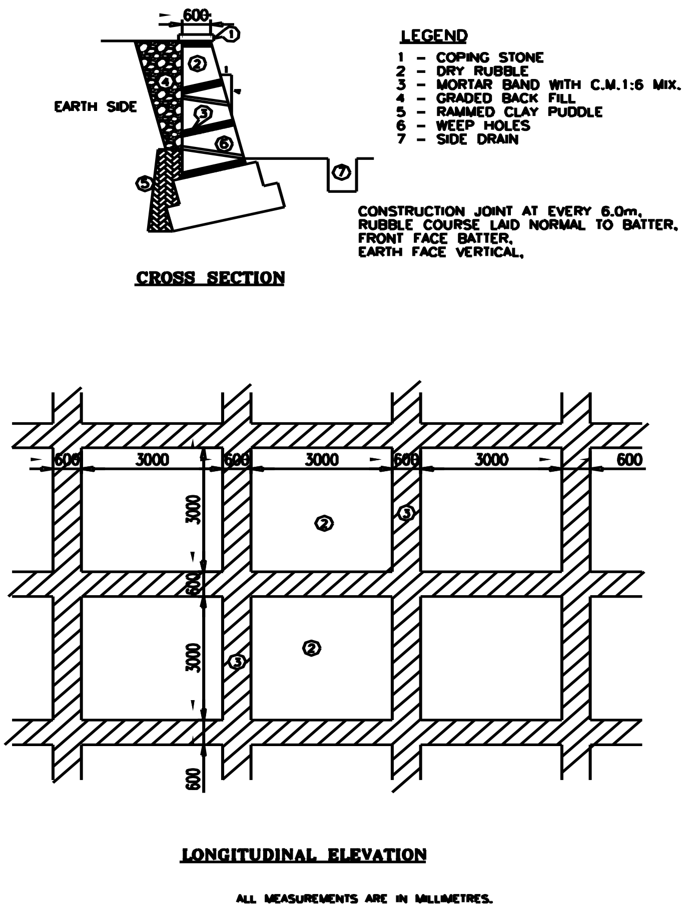


FIG.2: GRAVITY TYPE DRY RUBBLE BANDED WALL

7.2 Following are special features:

(i) Un-mortared dry joints of stones offer free drainage and frictional bond between stones whereas mortared bands offer stability.

(ii) It is a gravity type wall, designed for no tension condition.

(iii) Adequate frictional bond between stone layers from face to back and from base to top so that section of wall acts integrally as one unit. This can be achieved by using flat size stones of sufficiently large size providing interlocking and sufficient overlap.

(iv) Front face is batter, generally 1 in 4 and rear face is vertical. Top width is 0.6 – 0.75 m. As per prevailing practice, front batter is 1 horizontal to 4 vertical upto 12 ft height, 1 horizontal to 3 vertical upto 20 ft and 1 horizontal to 2.5 vertical for more than 20 ft height of wall. Base width can be arrived at accordingly.

(v) Masonry course shall be laid normal to face of batter and foundation also shall be sloping. Sloping course offers better stability. Dowels shall be provided in case rock is met with at foundation level.

(vi) Mortared bands are 0.6 m wide with 3 m to 4 m apart generally and lesser spacing, if higher stability is required.

(vii) Back of wall shall be rough. Back fill consists of 0.3 m thick rubble. To reduce the earth pressure on the wall and to improve drainage, highly frictional material can form the back fill with sizes graded and well compacted. Back fill grading shall be such a way that water only can pass through pores and not the cutting soil. Geo filters may also be provided.

(viii) Coping stone shall be placed on top.

(ix) To improve stability and to localize the slips, retaining wall shall be constructed for 6 m length only with clear gaps between two walls and more number of trolley refuges to be provided, say at every 50 m.

(x) To improve drainage, adequate number of weep holes to be provided of not less than 75 mm square. Rammed clay puddle shall be provided below the bottom most row of weep holes to prevent under mining of foundation.

(xi) Stones shall be large in size and have adequate weight to offer stability. This aspect is very important. It is a general practice that stone size is not less than 15 cm in any direction and bed width is more than 1.5 times the thickness. Stones shall be laid in interlocking pattern duly breaking the vertical joint between successive layers. Larger stones shall be placed in lower layers and stones shall be selected in such a way that contact area between stones is maximum to ensure higher frictional bond between stones.

8. MAINTENANCE PRACTICES

8.1 Following are the good maintenance practices for long term stability of cuttings. Pre-monsoon precautionary measures shall also be followed to avoid possible interruption to traffic during monsoon:

(i) Check on functioning of catch water drain, side drain and weep holes. Weep hole shall be cleaned with air jet and back fill shall not be clogged. It shall be freely draining.

(ii) To improve the drainage properties of backfill, hand held augers can be used to scoop out clogged earth and replace with permeable and highly frictional material.

(iii) Land use shall be monitored and misuse shall be prevented. Common misuses are plying of vehicular traffic on top of cutting, forming heavy duty roads, unauthorized cultivation, draining out water into cutting from houses, buildings, canals nearby. Inundation of water due to plantation shall be prevented. Trees like rubber, plantain, coconut, tapiaco etc require ponding of water which will impair stability of cutting. People may construct wells nearby for which blasting may be resorted to. This will trigger slips in cutting. Nearby quarrying may also trigger slips. Lined chutes in flatter slope are safer way to drain out water from top into cutting, if it is inescapable.

(iv) Sudden drawdown as well as impounding of dams / reservoirs nearby may cause slips.

(v) Breaches in canals or Railway affecting tanks nearby may also cause instability of cutting.

(vi) Large scale deforestation may affect moisture equilibrium.

(vii) Trees like Acacia, Manjanatha etc may affect moisture equilibrium drastically. Root thickening while growing on the slopes, open up the natural cleavages in the strata which may cause instability.

(viii) In North-South alignment and terrain sloping towards sea to South, East facing cuttings are more vulnerable to slip failures as per failure pattern.

(ix) Following monsoon reserve shall be kept:
chain links for anchoring under tension for protecting slope
Jute/coir net
Rails for shoring
Sand bags for slope protection
Pumps – Water pumps and mud pumps (slurry)
Plastic sheets / tarpaulins to prevent direct rainfall on slopes
Ready agencies for removal of slipped earth and temporary protection works.

(x) Most vulnerable cuttings can be instrumented with Acoustic emission technique to predict landslips. Longitudinal crack on top of cutting is an indicative of potential slip and can be watched by keyman.

9.0 RETROFITTING MEASURES

9.1 Slips and subsidence are possible due to various reasons. Downward movement of slope forming material along slope is called land slide whereas landslips are downward and outward movement of soil in an arc form. Slope failure occurs within the depth of slope whereas base failure occurs with the movement of base of slope. Ground subsidence occurs due to sudden changes in water table level which results in slumping of ground. Rainfall is the potential agent to encourage landslides and subsidence and intensity of rainfall contributes for devastating landslides. Drainage is the only effective solution for temporary and permanent restoration measures.

9.2 Temporary restoration measures are:

(i) Removal of slipped earth blocking the line.

(ii) Slopes pitched with sand bags with mouth tied and placed in interlocking pattern; this temporary shoring is a well established and highly successful practice ; gunny bags are found to be more effective than polythene bags in drainage.

(iii) Boulders precariously hanging shall be strutted in position with tensioned chain links as removal of one boulder may result in chain action of unstable hanging of other boulders. Use of tensioned wire ropes and rock bolting / strapping are also efficient solutions in strata with overhang of large size boulders. To prevent water ingress and create more instability, tarpaulins or plastic sheets shall be spread on slopes.

(iv) Improve drainage with perforated PVC pipes embedded in slopes and cleaning of weep holes.

(v) Removal of overburden during monsoon may cause heaving and may also expose soft soil below which is more unsafe. Moisture equilibrium shall not be suddenly disturbed to trigger further land slides.

(vi) Open excavation at toe during monsoon is unsafe. Toe shall be well protected and freely drainable.

(vii) Driving rail piles in monsoon may build up pore pressure in surrounding soil and may affect in situ equilibrium of surrounding soil. It may also cause lateral movement in clay soils. Rail piling shall be resorted to only if ground conditions are favourable.

9.3 **Permanent remedial measures are**:

(i) Reducing depth of cutting with due care on heaving.

(ii) Flattening side slopes with land acquisition; provision of berms and reprofiling.

(iii) Reconstruction / construction of retaining wall with dry stones or flexible retaining wall with reinforced earth gabions.

(iv) Reconstruction / construction of catch water drains.

(v) Dry stone pitching the slopes.

(vi) Drainage improvements and intercepting drains to prevent water entering into cutting zone. High pore water pressure in a slope can be decreased through horizontal or inclined drain holes. The drill holes are generally lined with slotted or perforated metal or plastic pipes. Vertical sand drains in clay with adequate diameter and spacing can also appreciably reduce pore pressure. Vertical exploration borings can also be converted into pumping bore wells for subsurface drainage.

(vii) Reducing earth pressure by improving the backfill – compacted backfill with granular, freely draining and high frictional angle.

(viii) Soil nailing – driven or bored nails.
(ix) Plantation which can bind the soil particles including turfing.

10.0 STATE-OF-THE-ART MATERIALS FOR SLOPE PROTECTION

10.1 Apart from natural stones, coir and jute for slope protection, following family of synthetic materials called Geo-synthetics are found to be efficient.

Geotextiles:	drainage and filtration
Geogrid:	soil reinforcement
Geomembrane :	to control fluid migration / barrier for flow of water
Geonet/Geomesh :	drainage
Geomat/Geoweb:	erosion control, admixtue to strengthen soil
Geocomposite :	Combination of the above.

10.2 Geotextile wrapped French drains have several advantages over conventional open drains and gravel filled trench drains by preventing significant particle movement from the adjacent soil into the drainage and providing sufficient hydraulic conductivity to permit free flow of water into the drains.

10.3 Geogrid reinforced gabion walls have the advantage of reinforcing the fill soil with geogrid and improving drainage through filling of graded stones inside gabions. By properly choosing the backfill with highly frictional material, shear resistance to sliding can be greatly enhanced and earth pressure on gabion can be significantly reduced. Typical arrangement is shown in Fig.3. This proposition calls for wider excavation width to accommodate reinforcement length and gabion width than conventional excavation width for rigid retaining walls. Cost and safe excavation aspects are to be examined before finally adopting.

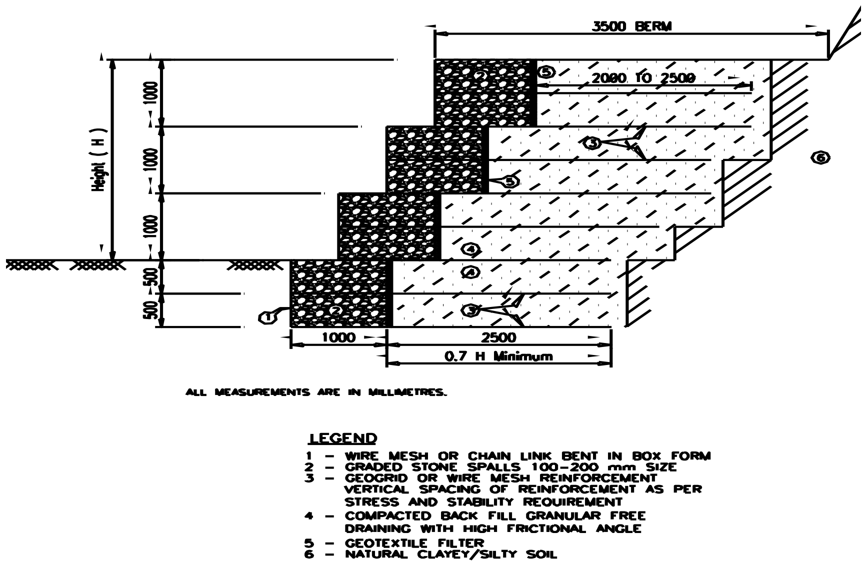


FIG.3: FLEXIBLE RETAINING WALL (TYPICAL)
(REINFORCED EARTH GABIONS WALL)

10.4 Anchored spider netting with geonet/geogrid spread over slope and anchoring locally at points with long steel rod nails radially is an effective solution for reinforcing the soil in the slopes. Typical arrangement is shown in Fig.4. These nails must be long enough to penetrate potential failure surface and nets shall be surface tensioned to have compressive influence on enclosed soil mass. Nails are to be re-driven in case tension is lost gradually due to soil creep or seepage over a period of time.

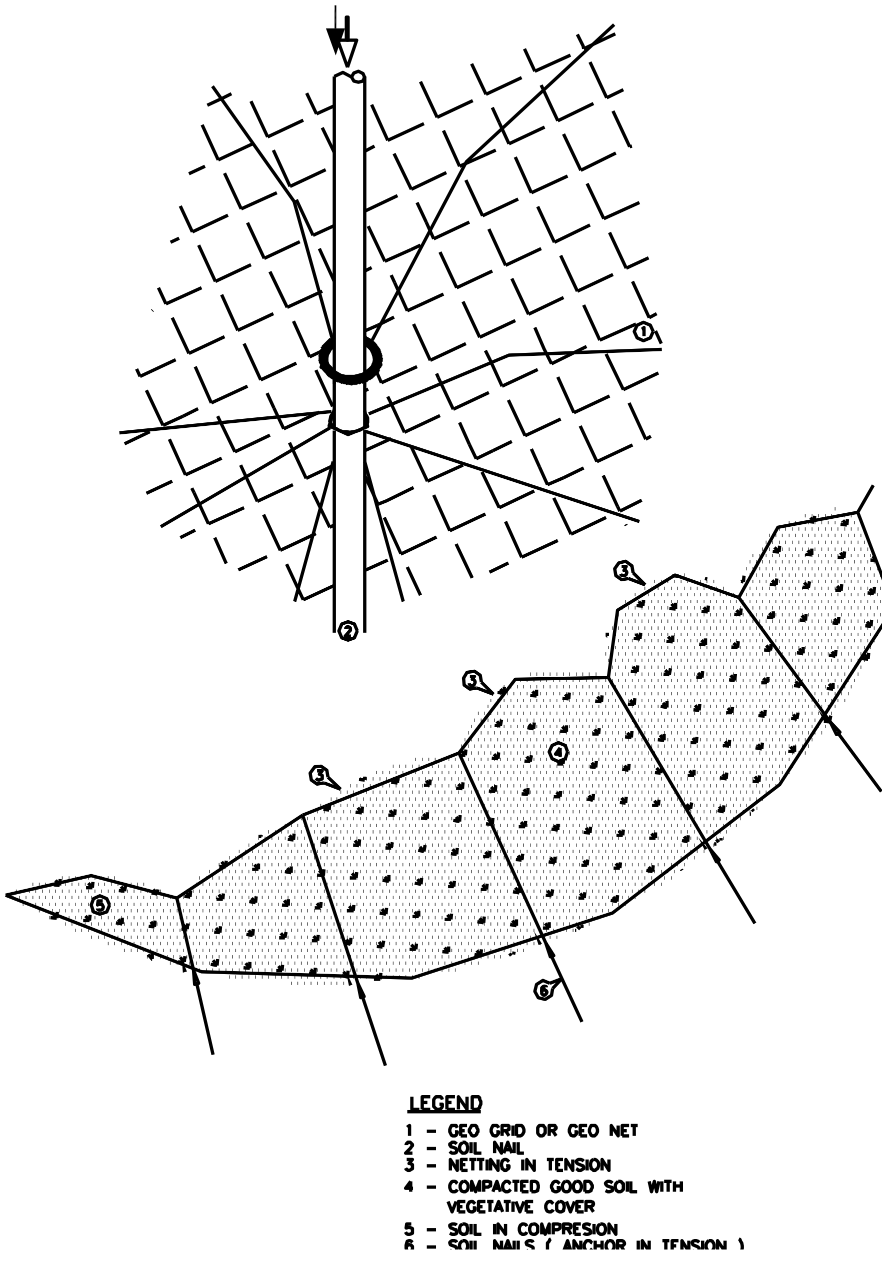


FIG.4: ANCHORED SPIDER NETTING

11.0 Photographs depicting good construction practices adopted for Chala cutting between Tellicherry and Cannanore in West Coast line which has withstood rainfall of 100 year return period are annexed.

CHAPTER VIII

CONSTRUCTION & MAINTENANCE ASPECTS OF CUTTINGS

A. CONSTRUCTION ASPECTS

1.0 SURVEY AND EXPLORATION

1.1 The object of detailed exploration that includes surface & sub-surface exploration is to ascertain incipient failure mechanism and gather necessary data required for the design. With this objective in view, exploration should include

- Determination of depths of various soil strata and rock beds.
- Evaluation of nature of soil and rock and their engineering properties that may affect the design and construction of structures.
- Ascertaining the site geology
- Ground water table, its seasonal variation and its effects.
- Chemical properties of rock, presence of discontinuities, zones of weakness and water wherever warranted.
- Dip, strike and hardness of rock
- Folding or bending of rock
- Joints or fracture of rock
- Faulting plane or fracture zone of rock
- Rainfall, ground slope and surface discharge data
- Erosion potential of alignment including susceptibility of alignment to damage/erosion by streams and torrents
- Effect of project construction on adjacent and nearby streams, ponds and lakes
- Study of existing cuttings, their behaviour and successful protection works, if any

1.2 When construction of Railway formation requires rock excavation and slope design, it is imperative that adequate in-situ and laboratory information is obtained on the structural nature of the rock. Beyond helping in making sound design, preliminary information will provide realistic parameters for estimating both construction cost and schedules.

1.3 The nature of structural discontinuities in the rock mass should be identified, as it will govern ultimate stability of rock cut formation. Major rock cuts will typically require detailed sub-surface exploration where as minor rock cuts may often be designed by an experienced engineer without sub-surface exploration.

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1.4 The release of horizontal stresses during excavation in soft rock may cause the formation fissures. If water enters the fissures, the strength will decrease progressively, therefore, the long-term stability of slope in soft rocks is normally more critical than short term stability.

1.5 An increase of ground water table results in a decrease of frictional resistance in soft rock and swell in cohesive material. Where highest water table is at or proposed formation level, suitable measures would be necessary to drain water out of the cutting and keep the sub-grade reasonably dry.

2.0 PLANNING

Following aspects should be given due consideration in planning stage: -

2.1 While selecting the alignment, unstable strata/landslide prone areas /erosion prone areas should be avoided as far as possible. For this purpose, geological maps should be studied and the advice of geo-technical engineers, geologists, forest and soil conservation experts should be taken right from the start.

2.2 During the process of construction, cut and fill method should be resorted to in order to cause minimum disturbance.

2.3 The alignment should avoid large-scale cuttings and fillings, and follow the profile of land as for as possible.

2.4 To facilitate the survey team in the tentative selection of alternative alignments for subsequent detailed ground reconnaissance or for estimation of present health of hilly terrain, it will be advisable, if economy of projects permits, to take advantage of modern techniques like Aerial Survey, Photogrammetry and Remote Sensing.

2.5 In high mountain ranges, it may be expedient and economical in some cases to construct tunnels to shorten the length of the alignment.

2.6 Use of tunnels to avoid deep cuts more than 25 m should be considered where feasible and economical. Deep cuts involving destabilization of natural hill slope should be avoided.

2.7 In hilly country, a location subject to sunlight should get preference over a location in the shade.

2.8 To the extent feasible, alignment should be away from streams and torrents except where these are to be crossed.

2.9 Rock blasting must be properly planned and controlled. Rock blasting work for a project must be planned in detail at the stage of preparation of estimates. Heavy rock blasting should be avoided and controlled blasting should be resorted to.

2.10 Proper disposal of surplus excavated material should be thought of and provided for. Spoil from cut/blasted rock should not be thrown haphazardly along the valley slopes but dumped duly dressed in a suitable form at suitable places where it cannot get easily washed away by rain as these are likely to cause heavy siltation/chokage of water channels/streams and damage agricultural lands. In side hill cuts, material excavated from the upper slope should never be cast over on the downhill shoulder. This practice promotes failure of the downhill slope. Such excavated material should be loaded and taken away.

2.11 The design cut slopes should be stable for the type of strata. Cut slopes should be rendered stable in the construction stage itself by cutting at the correct angle and benching etc. including slope stabilizing and protection measures. All slope protection & stabilizing works and other erosion control items should be identified on the plans and provided in the proposals. Wherever considered appropriate on the basis of a technical study conducted for the purpose, funds should be provided in the project estimates for the treatment of the unstable areas.

2.12 Drainage of water should be given adequate attention and an effective system of drainage should be constructed to lead the run-off to natural water courses. In particular, suitable intercepting and catch water drains should be provided above the cut slopes for the speedy and safe disposal of water. It should be ensured that water is not drained into villages and cultivate land. Location and alignment of cross drains and culverts should be so chosen as to avoid erosion of the outlet and silt deposition at inlets. The necessary erosion control measures should be undertaken at the out falls of culverts along-with pitching/paving of the channel.

2.13 Soil maps & aerial photographs studies and investigations should be made to locate areas or sections with high erosion potential and project estimate should provide for the necessary measures against soil erosion.

2.14 The original vegetation will help to bind a slope together and provide subsurface drainage by transpiration. It should be preserved to the maximum extent possible on stable slopes. On freshly cut slopes, vegetation should be reestablished to prevent erosion. After erosion develops and gullies are formed, protection is more difficult to arrange.

2.15 Deforestation during the construction of road should be kept to the minimum and should be done only in consultation with Forest Authorities.

2.16 Construction of track in hilly areas is itself an encroachment on surroundings, disturbing natural state. As a balanced eco–system is essential for survival of all living species it becomes imperative that when track is developed in such region then preservation of environment and ecological balance must be a part of the project.

3.0 <u>DRAWINGS</u>

Drawings should be prepared to include following aspects:

3.1 A plan and profile should show the location of the route, significant features of the adjacent area, existing ground levels and the level of new grade. The drawing should also show road crossings, culverts, pipelines and utilities, location and water level of ponds and streams along the right of way, type of vegetation, and other information of use in planning the work. Rock & soil types, surface outcrops of rock and presence of boulders should be shown. The nature and arrangement of soil and rock deposits which will be encountered and their possible behaviour under the effects of weather variations and disturbance of construction traffic & operations should also be included.

3.2 Typical cross sections of cuts and fills showing all dimensions, slopes, and classification are required. Drainage excavations and culvert details should be shown as well as details of berms or benches, slope drainage, erosion protection, and stabilizing & protection measures.

3.3 Detailed logs of test pits and borings should be shown. Standard terminology should always be used. Results of standard tests should be included.

4.0 EXCAVATION

4.1 Open excavation refers to the removal of material, within certain specified limits, for construction purposes. For this to be accomplished economically and without hazard, the character of the rocks and soils involved and their geological setting must be investigated. Indeed, the method of excavation and the rate of progress are very much influenced by the geology of the site. Another factor which is important is the position of the water table in relation to the base level of the excavation, as are any possible effects of construction on the surrounding ground and buildings.

4.2 Groundwater frequently provides one of the most difficult problems during excavation work and its removal can prove costly. Not only does water make working conditions difficult but an appreciable, flow of water can lead to erosion and failure of the sides. This, of course, increases costs since not only does the collapsed material have to be removed but the damage has to be made good. Subsurface water is under pressure, which increases with depth below the water table to high values. Hence, data relating to groundwater conditions should be obtained prior to the commencement of operations.

4.3 Artesian conditions can cause serious trouble in excavations and therefore, if such conditions are suspected it is essential that both the position of the water table and the piezometric pressures should be determined before work commences. Otherwise excavations which extend close to strata under artesian pressure may be severely damaged due to blow outs taking place in their floors. Such action may also cause slopes to fail and could lead to the abandonment of the site in question. Sites at which such problems are likely to be encountered should be dealt with, prior to and during excavation, by employing dewatering techniques.

4.4 Following general points should also be kept in view during excavation-

4.4.1 No excavation work shall commence until the clearing, grubbing and prior drainage, where required, have been advanced to the satisfaction of the engineer.

4.4.2 Scaling and Trimming operations form the important precursor to the excavation of a rock slope. They would be carried out in the region above the excavation with the objective of minimizing rock fall during and after construction.

4.4.3 All excavation areas shall be completed as far as is practical before borrow materials are obtained from outside sources.

4.4.4 Where catch ditches are required at the top of slopes, they shall be excavated in advance of any adjacent cut excavation.

4.4.5 All working surfaces in cuts should be maintained in a well-drained condition at all times.

4.4.6 Side slopes in rock cuts may be formed by the general method of shaping them concurrently with or after the removal of material from the cut or by the method of advance presplitting rock along the required plane by blasting. Rock beyond the line of the side slopes which is loosened by blasting, rendering it liable to slide or fall in the opinion of the engineer, shall be removed.

4.4.7 Where rock materials are required for construction of fills, blasting should be carried out in such a manner that the rock will generally meet fill requirements.

4.4.8 The bottom of rock cuts shall be excavated in such a manner that there will be free drainage without water pockets.

4.4.9 Complete and continuous precautions shall be taken to prevent any damage to staff working and equipment such as vehicles, trains, power or communication lines, structures, private dwellings or other installations by reason of concussion, vibration or flying material.

4.4.10 All regulations governing the transportation, storage, handling and use of explosives at the location of the work should be complied and such permits as are required should be obtained from appropriate governing authority for use of explosives.

4.4.11 All necessary precautions should be taken against the effects of induced currents caused by radio transmitters and receivers, power lines, transformers, cables, radar beams or any other energy or wave force which might result in premature firing of the blasting circuits.

4.4.12 All blasting shall be done with extreme care by experienced and authorized men and adequately supervised by technical personnel, in accordance with procedures approved by the engineer in writing.

4.4.13 Selection of blasting holes should be so done as to avoid large scale disturbance to the rock face, developing cleavage planes/cracks and opening up fissures etc.

4.4.14 Before starting drilling operation, drilling & loading pattern for blasting should be approved by the engineer. All drill dust shall be blown out of the holes and the same shall be protected with suitable plugs or covers. Each blast shall be subject to clearance from the engineer.

4.4.15 When necessary to protect property or facilities, all blasts shall be suitably covered with blasting mats or other approved protective material, weighted and secured in such a manner as to prevent projection of debris.

4.4.16 When slides occur in cuts after they are properly formed, material shall be removed immediately, the slopes modified and such other precautions adopted for stabilizing slopes.

4.4.17 Daily account must be maintained of rock blasting to be done, explosives planned/expected to be used and actually used under the direct control of the Engineer and wherever there is any anticipated deviation in quantities from that planned, it should be checked.

4.4.18 Blasted material and debris rolling down should be avoided. Subject to cost effectiveness, the debris should be moved to selected safer places where these are not likely to be washed away.

5.0 METHODS OF EXCAVATION

5.1 Ripping

5.1.1 Ripping is an inexpensive method of breaking discontinuous ground or soft rock masses, the fragmented material being removed by bulldozer. Rippers used to break hard soil are pulled by tractors and consist of rugged frames into which are fitted one or more steel tynes. These are held down into the ground whilst being drawn through it, thus causing the ground to break.

5.1.2 Rippability in rock masses depends on their intact strength, fracture index and abrasiveness. In other words, strong massive and abrasive rocks to not lend themselves to ripping. On the other hand, if sedimentary rocks such as sandstone and limestone are well-bedded and jointed or if strong and weak rocks are thinly interbedded, then they can be excavated by ripping rather than by blasting. Indeed, some of the weaker sedimentary rocks having -less than 15 MPa compressive strength or 1 MPa tensile strength such as mudstones are not as easily removed by blasting. They are pulverized in the immediate vicinity of the blast-hole. What is more, blasted mudstones may lift along bedding planes to fall back when the gas pressure has been dissipated. Such rocks, especially if well jointed, are more suited to ripping than blasting.

5.1.3 The run direction during ripping should be normal to any vertical joint planes, down-dip to any inclined strata, that is, normal to the strike, and on sloping ground it should be downhill. Adequate breakage in rock depends on the spacing between ripper runs which in turn is governed by the fracture pattern in the rock.

5.1.4 The output of a ripper is influenced by the incidence and size of discontinuities and the strength of the rock. It also depends upon the capacity of the bulldozer. Output generally falls within the range of 40 to 230 m^3/h.

5.1.5 The most common method for determining whether a rock mass is capable of being ripped is seismic refraction. The seismic velocity of the rock concerned is compared with a chart of ripper performance based on ripping operations in a wide variety of rocks. In fact, the limit of ripper operations can be regarded as a seismic velocity of 2 km/s. However, the ground can be loosened prior to ripping by hydraulic fracturing or blasting if the strength of the rock mass is high or if ripper production has to be increased. Such a pre-blasting process uses light charges to open the discontinuities.

5.2 <u>Hydraulic fracturing</u>

5.2.1 Hydraulic fracturing the process whereby rock is broken by applying water at high pressures in drill holes. The use of this technique allows the rock to be fractured at depths between 0.5 and 1.5 m. A high-speed rotary percussion drill is used to sink the holes in which fracturing occurs but, before fracturing commences, the holes are filled with sand.

5.2.2 The machine used for hydraulic fracturing is effective in rock in which the direction of the discontinuities is within 45° of the horizontal. The discontinuities should be closely spaced and their apertures should not be wide. By contrast, if the rock mass contains open, randomly distributed discontinuities which are more or less perpendicular to the ground surface, fracturing is not very successful because the water pressure is too readily dissipated.

5.3 <u>Drilling and blasting</u>

It involves drilling with specific equipment, blasting with explosives and the clearance of blasted debris with dozers. These, being very expensive and risky operations, call for thoughtful planning and careful execution by personnel having thorough knowledge and extensive practical experience in rock cutting work and use of drilling and blasting equipment and explosives. Apart from the immediate cost and risk, excessive or improper use of explosive may result in large scale disturbance of hill side creating slide areas leading to erosion and expensive control measures.

The planning of rock cutting work comprises of

- determination of resources/stores required based on estimated quantum of work, output norms and target time for completion of work.

- location of most advantageous points/ faces to commence the work based on detailed ground reconnaissance.
- working out the drilling pattern most suitable to the particular location.
- efforts should be to collect and store as much drilled rock as possible to use the same for subsequent protective drainage and pavement works.

Drilling and blasting, although generally the most effective and economical method of excavating rock, are not desirable in built-up areas since damage to property or inconvenience may be caused. Neither is it wise to blast where landslides or rockfalls might result.

5.3.1 **Drilling**

The properties of a rock mass which influence drillability include strength, hardness, toughness, abrasiveness, grain size and discontinuities. In addition to the properties of the rock mass, the penetration rate is influenced by the power of the rock drill, the shape of the cutting edge of the bit, the air pressure at the rock drill and the diameter of the drillhole. The type of flush used and the experience of the drilling crew also influence the rate of penetration.

Percussion type churn drills penetrate the hardest rocks but they are slow and inefficient. Consequently, they have been largely superseded by down-the-hole and rotary drills, since the penetration rate of both is much faster. In softer rocks, rotary drills give faster penetration than the percussive types but in hard to medium rocks, rotary drill is not economical because of the severity of bit wear. The best compromise is the rotary percussion drill. Blast holes of up to 50 mm diameter can be sunk in soft to medium hard rocks e.g. some sandstones, shale, coal, gypsum and rock salt) with rotary drills.

Rotary percussion drills are designed for rapid drilling in rock, they provide a continuous rotary cutting action upon which axial percussive blows are superimposed. In this way, the rock is subjected to rapid high-speed impacts whilst the bit rotates, which cause fracture at a lower value of rotary thrust and torque. Consequently, faster rates of penetration are achieved at lower thrust than with rotary drilling. The technique is most effective in brittle materials since it relies upon chipping the rock.

The impact mechanism is coupled to the bit in a down-the-hole hammer drill and accompanies the bit down the hole. Generally, compressed air is used with a down the hole hammer.

Where the depth of excavation does not exceed 3.5 m, the use of jack hammers to drill holes achieves good results. Mounted drilling machines are required for deeper excavations.

The dip of a drill hole can be measured with an inclinometer. If the deviation is excessive, then this will entail redrilling. Generally speaking, the possibility of deviation tends to increase with distance from the top of the hole and its incidence tends to be greater in angled than in vertical holes.

5.3.2 Blasting Operations

One of the fundamental requirements of blasting is that a free face should exist in front of the blast holes when they are fired. When the explosive in a blast hole is fired, it is transformed into a gas, the pressure of which may sometimes exceed 100000 atmospheres . The tremendous energy liberated shatters a zone around the blast hole and exposes the rock beyond to enormous tensile stresses. This takes place under the influence of shock waves which radiate from the explosion at between 3000 and 5000 m/s. A zone of intense deformation occurs about the blast hole, its thickness frequently approximating to the diameter of the hole, and radial cracking extends appreciably further.

Under the influence of the pressure of gases from the explosive, the primary radial cracks expand and the free face yields and is moved forwards. When this occurs, the pressure is unloaded and tension increases in the primary cracks which incline obliquely forward. Several of these cracks expand to the exposed surface and the rock is completely loosened. In this way, the burden is torn from the rock mass. The lateral pressure in the shock wave is initially positive but falls rapidly to negative values implying a change from compression to tension. Hence the area beyond the hole is exposed to vast tangential stresses. The shock wave itself is not responsible for breakage of rock but it does provide the basic conditions for the process. Shock wave energy when using high explosive may only account for 5 to 15% of the total energy liberated.

The discontinuities within a rock mass act as free surfaces which reflect shock waves generated by an explosion. They also provide paths of escape along which energy is dissipated. The geometry of the discontinuity pattern is very important since the greatest loss of energy occurs where most discontinuities intersect.

Rock breakage in blasting, apart from the character of the rock itself, especially the fracture index, depends largely on the relation between the burden and the hole spacing and also on the time of ignition between the holes. Efficient blasting should produce rock fragments sufficiently reduced in size so that they can be easily loaded without resort to secondary breakage. Accordingly, blast holes should be drilled accurately to the requisite pattern and proper depth to ensure satisfactory fragmentation. The greater the capacity of the buckets of the loading machines, the larger the fragment size that is acceptable. Good fragmentation also reduces wear and tear on loading machinery.

Though there are a number of expressions relating burden and spacing of holes, however, careful trials provide the only means of determining the burden and blasting pattern in any rock. Generally, the spacing will vary between 0.75 to 1.25 times the burden.

The quantity of a particular explosive required to blast a certain volume of rock is difficult to estimate since it depends upon the strength, toughness and incidence of discontinuities within the rock. There needs to be a higher concentration of charge at the base than in the column, whilst at the top of the hole no charging is required. Hence the concentration of explosive in the bottom of the hole generally is approximately 2.5 times greater than in the column. The basal charge may be regarded as extending from the bottom of the hole to a point above floor level equal to the burden. The column charge extends from there to a point below the crest equal to the burden.

The following conditions should be satisfied if the blast is to be optimized and damage to the rock behind final row minimized-

- The front row charge should be adequately design to move the front row burden.
- The main charge and blast hole patterns should be optimized to give the best possible fragmentation and digging conditions for the minimum powder factor.
- Adequate delays should be used to ensure good movement towards free faces and the creation of new free faces for the following row.
- Delays should be used to control the maximum instantaneous charge to ensure that rock breakage does not occur in the rock mass which suppose to be remain intact.
- Bach row holes should be drilled at an optimum distance from the final dig line to permit free digging and yet minimize damage to the wall.

5.3.3 <u>Explosives for blasting operations</u>

The essential characteristic of an explosive material is that, on initiation, it reacts suddenly to form large volumes of gases at high temperatures, the almost instantaneous release of these gases giving rise to very high pressure. This reaction proceeds very rapidly and is self sustaining in that it continues throughout the explosive when it is set off at any point within it.

Initiating explosives are used to detonate high explosives and as such, they are extremely sensitive and relatively easy to explode. Small quantities of initiating explosives are contained in copper or aluminium tubes, thereby forming detonators. When ignited, they produce an intense local shock which starts the reaction in less-sensitive high explosives. Initiating explosives do not necessarily produce large volumes of gases and alone are not suitable for blasting operations.

High explosives detonate at velocities from about 1500 to 7500 m/s, depending upon the explosive composition and large volumes of gases are formed at exceptionally high pressures. The reaction is started and sustained by a shock wave initiated by a detonator. The rate at which the detonation wave travels through a column of explosives is termed the velocity of detonation. The shock wave developed by the explosive is transmitted through and produces fractures in the rock mass concerned. The movement of the high pressure gases through the rock and mass completes its breakage. The performance of a high explosive depends on the velocity of detonation and on the volume and temperature of the gases which form. Its power is governed by the amount of energy released when it is detonated.

Charges are decked by separating zones of the explosive column by using inert materials, that is, stemming. The peak blast hole pressure within each charge deck is not reduced but the rate of decay of pressure can be increased appreciably. Because of availability, drill hole chippings are most frequently used in stemming. In dry blast holes, chippings, angular crushed rock (around 15 to 25 mm in size for holes with diameters between 225 and 380 mm) is better than chippings. But even crushed rock is not as good as air decks in this respect. These are constructed by locating closed rigid empty cylinders between the charges in the blast hole.

Plain detonators and safety fuse, electric detonators (instantaneous and short delay), Cordtex detonating fuse and detonating relays may be used for initiation. Because remote control is easy, electric shotfiring is used for most operations. DO's & DON'Ts on the use of explosives are given in Annexure-III.

6.0 TERMINOLOGY

6.1 Explosives

An explosive is a substance or a mixture of substance, which for the purpose of transport, handling and storage is in stable equilibrium. The equilibrium is upset, if subjected to severe shock resulting in violent release of energy in the form of shock waves accompanied by extremely rapid conversion of the explosives into a large volume of gases at high temperature and pressure. The necessary shock is provided by means of a detonator or detonating fuse. The choice of explosives and accessories depends upon the nature of strata and its characteristics.

6.2 Power

The most important property of an explosive is its strength or power. Blasting gelatine, the most powerful of all commercial explosives, is taken as the standard and the power of all other explosives are measured in relation to the power of blasting gelatine, indicated as percentages.

6.3 Velocity of detonation

It is the rate at which the detonating wave travels through a column of explosives. Shock energy of detonation increases rapidly with velocity.

6.4 Density

The density is important when selecting an explosive for a particular use. With a high density explosive, the energy of the shot is concentrated whereas a low density explosive distributes energy along the shot hole.

6.5 Water resistance

Explosives differ widely in resistance to water and moisture penetration. While some explosives deteriorate rapidly under wet conditions, others are designed to withstand water for considerable periods. If blasting is to be done under wet conditions, a water resistant explosive should preferably be selected.

6.6 Sensitivity

An explosive must not be sensitive to normal handling, shock and fraction, but it must remain sufficiently sensitive to be satisfactorily detonated and capable of propagating satisfactorily, cartridge to cartridge even over short gaps, such as may occur in practice.

6.7 Fume characteristics

Explosives when used under stipulated ventilation conditions should liberate minimum of harmful gases in the process of detonation.

6.8 Thermal stability

Explosive combination should be such as to be stable under all normal conditions of transportation, handling and usage. This is extremely important as use of explosive itself depends on this characteristic.

6.9 Detonator

Intense local shock for initiating high explosives is produced by a detonator. They have extremely sensitive composition loaded in small quantities into copper or aluminium tubes. It is the spark or SPIT from a safety fuse, which causes the detonator to explode. A variety of detonators are used for blasting.

6.10 Plain detonators

Plain detonator consists of a small aluminium tube closed at one end and it is used with safety fuse. It contains a base charge and a priming charge.

6.11 Ordinary electric detonators

An ordinary or instantaneous electric detonator is essentially a plain detonator plus a fuse head coupled to a pair of leading wires and is triggered by electric current.

6.12 Delay detonators

This detonator consists basically of an electric detonator with appropriate delay element interposed between the fusehead and the priming charge. They are used when blasting operations call for a series of shots to be fired in a pre-determined sequence.

6.13 Detonating fuse

Detonating fuse is a simple and safe device for initiating cap sensitive commercial explosives particularly suitable for simultaneous firing of multiple charges and for the mass initiation of large charges. It consists of a core of PETN(PENTA ERYTHRTOL TETRANITRATE) with covering of textile and plastic. It is initiated by a detonator and detonates at a velocity of approximately 6500 m/sec.

6.14 Safety fuse

Safety Fuse consists of a thin core of specially prepared black powder wrapped in layers of textile yarn and waterproof coating. The burning speed of safety fuse is controlled.

6.15 Exploder

When explosives are to be initiated electrically, a portable exploder for generating electricity is used. An exploder, is generally built on a metal chassis contained in a waterproof 'bakelite' case. It comprises of a dynamo which is operated by turning a handle. The A. C. Voltage generated by the dynamo is stepped up by a transformer rectifier and used to charge a condenser to a potential of not less than 1,200 volts. When the firing button is pressed, the condenser is discharged through the electric circuit firing the shots.

6.16 Benching

A 'bench' is a free face of the rock developed for ensuring effective utilization of explosives.

6.17 Burden

'Burden' is the perpendicular distance from shot hole to the nearest free face of the rock in the direction in which the displacement is most likely to occur. Its actual value will depend upon a combination of variables including rock characteristics, the nature of explosive and the diameter of explosive etc.

6.18 Depth of hole

The depth of a drill hole depends upon type of drilling equipment and loading method adopted. As a rule, the depth of hole should never be less than the 'Burden'. In practice holes are drilled to a depth varying from 1.5-2.5 times the 'Burden'. While deciding the depth of drill holes, it must be borne in mind that depth of stemming should not be less than the 'Burden'; otherwise Line of Least Resistance will be established in the direction of stemming and blasting may not be much effective.

In general, a few deep bore holes are more efficient than large number of shallow ones as the amount of material detached is proportional to the cube of the depth of the charge. Also, the vertical holes are easy to drill and normally give best results.

6.19 <u>Spacing</u>

Spacing is the distance between boreholes or charges in the same row. It is dependent upon the nature of rock, degree of fragmentation required and the method of firing. Where holes are fired singly or with large intervals, the spacing may be twice the 'Burden'. However, where shot holes are fired simultaneously, which is the most common practice, spacing should not exceed 1.5 times the 'Burden'. In very hard and tough rocks, the spacing may have to be less than the 'Burden'. The optimum spacing between drilled holes should be determined by trials.

6.20 <u>Loading Density</u>

Loading density of explosive means the weight (in kg) of explosive charge per m of bore hole. It is different for different hole diameter and explosives due to varying densities.

6.21 <u>Volume of rock blasted</u>

The volume of rock blasted is proportional to the depth and spacing of drill holes and burden and is given by the formula :

Volume of rock blasted per hole (Cum) = Depth (m) x Total length of face (m) x Spacing (m)

For jack hammer holes it is not necessary to calculate the volume per individual hole. The total volume to be blasted may be calculated as follows :

Total volume of rock (Cum) = Average depth (m) x Total length of face (m) x Average burden (m)

6.23 <u>Blasting ratio</u>

The volume of rock broken by a unit weight of explosive is known as "Blasting Ratio". This ratio is usually expressed in "Cum of rock broken per kg of explosives".

6.24 <u>Quantity of explosives required</u>

The quantity of explosives required per hole or per blast can be worked out as follows :

$$\text{Quantity of explosives (kg)} - \frac{\text{Volume of rock (cu.m.)}}{\text{Blasting ratio (cu.m.)}}$$

7.0 TRANSPORTATION OF EXPLOSIVES

7.1 All the relevant central, state and local laws, rules & regulations framed thereunder shall be complied. Loading, unloading and handling of explosives shall be supervised by qualified personnel. At the time of loading or unloading of explosives, no electrical switch should be operated.

7.2 Containers

For carrying small quantity (upto 5 kg of explosives) specially designed insulated containers may be used. These containers shall be constructed of finished wood not less than 50 mm thick or plastic material not less than 6 mm thick or pressed fibre not less than 10 mm thick. Metal components, including nails, bolts, screws, etc., shall not be used in the construction of the containers, which shall be waterproof and provided with lids. The containers shall be provided with suitable non-conductive carrying device, such as rubber, leather or canvas handle strap.

7.3 Vehicles

The vehicles used for transporting explosive shall be driven only by an experienced driver who is physically fit and is familiar with the precautions to be taken while carrying the explosives in the vehicle. All vehicles used for transporting explosives shall be maintained in good working condition and all systems of same must be checked before starting the vehicle. The vehicles should preferably be enclosed type with locking arrangements and body-work leak-proof.

7.3.1 In open body vehicle, the floor of the vehicle carrying explosive shall be leak proof. The sides and ends shall be of sufficient height to prevent the explosive from falling off the vehicles.

- The interior of the body shall not have any exposed metal parts, except those of copper, brass and other non-sparking metals and shall preferably be lined with wood.

- The chassis of the vehicles shall be well sprung. The tyre pressure shall be maintained as per the requirement of the Indian Explosives Regulations.

7.3.2 All electrical wiring and equipment of vehicles shall be adequately insulated and protected against mechanical damage to prevent short circuiting.

7.3.3 Two carbon dioxide fire extinguishers, each of not less than 3 kg capacity, conforming to IS: 2878-1986, shall be carried on each vehicle. The extinguishers shall be securely mounted on the vehicles in such a manner that they can be readily removed for use in an emergency.

7.3.4 A motor vehicle carrying explosives shall not be refuelled except in emergencies and even then only when the motor has been stopped and other precautions have been taken to prevent accidents.

7.3.5 The quantity carried, in any single vehicle should not exceed 75 % of its rated capacity or 3600 kg, whichever is less.

7.4 Safety Precautions in Transportation

7.4.1 No metals except approved metal truck bodies shall be allowed to come in contact with cases of explosives. Metal, flammable or corrosive substances shall not be transported with explosives. As far as possible, transportation of any other material along-with explosives shall be prohibited.

7.4.2 Smoking shall be prohibited in the vehicle carrying explosives and in its vicinity upto a distance of 30 m.

7.4.3 No unauthorised person shall be allowed in the vehicle carrying explosives

7.4.4 Explosives and detonators of blasting caps shall not be permitted to be transported in the same vehicle.

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7.4.5 Detonators and other explosives for blasting shall be transported to the site of work in the original containers or in securely locked separate non-metallic container and shall not be carried loose or mixed with other materials.

7.4.6 Care shall be taken while loading and unloading of explosives, like inside of vehicle body must be free from grit, oil rags etc., unloading should not be done near exhaust of pipe, explosive protected from rain/prolonged exposure to sun, engine of vehicle switched off and no refuelling permitted while unloading etc. The filled containers shall not be handled roughly or dropped.

7.4.7 Drivers shall not leave the vehicles unattended while transporting explosives.

7.4.8 The speed of the vehicle shall not exceed 25 km/h on rough roads and 40 km/h elsewhere.

7.4.9 Vehicles, transporting explosives shall not be taken into a garage, repair shop or parked in congested areas, public parking or similar places.

7.4.10 Explosives shall not be transported in trailers. Further, any trailer shall not be attached to a motor truck or vehicle when it is being used in transporting explosives.

7.4.11 Explosives shall not be transported on public highways during darkness, except in emergencies and even then only when the approval of the concerned authorities has been obtained. Such vehicles shall be fitted with adequate warning lights on both ends, while operating in darkness.

7.4.12 Explosives shall not be transferred from one vehicle to another on public highways, except in cases of emergency.

7.4.13 When explosives are carried in a convoy, the distance between any two vehicles will not be less than 75 m.

8.0 STORAGE OF EXPLOSIVES

8.1 Explosives are to be stored in a 'Magazine' only and in accordance with provisions of the rules made under the Indian Explosives Act, 1884. Explosive Magazines can be of Mode 'A' or Mode 'B' type. Mode 'A' magazines are of permanent construction and Mode 'B' are the portable magazines. These are approved by the Chief Controller of Explosives.

8.2 Magazine should be clean, dry, well ventilated, well illuminated where electricity is available, reasonably cool, correctly located (more than 100 m from living accommodation) and protected against lightening if explosive is one ton or more in accordance with Indian Electricity Act and Indian Explosives Act. The magazine should be located on well drained sloping ground and away from built-up area/highway but approachable with all-weather road. In case of new storage accommodation for explosive, the local inspector of explosive or other licencing authority should be consulted and care should be taken to ensure that the statutory distances from other buildings and property are observed.

8.3 All major dumps as well as dumps in disturbed area having explosive of 3 ton or more should be fenced with double fencing of barbed wire. Similarly, all precautions of security must also be taken for safe storage of explosives.

8.4 Explosive cases should not be stacked in more than five tiers and should be stacked in such a way that ends of the cases showing the date of manufacture are visible, which will facilitate use of stock early.

8.5 Explosives upto 4 kg should be kept in a securely locked container away from fire and detonators/capped fuses should be kept in separate containers. While storing explosive upto 20-25 kg, a small store/magazine should be built. In case of storage for large quantity, following guidelines be followed.

8.5.1 Building should be specially constructed for this purpose situated away from residential/industrial area and highway.

8.5.2 About 2.42 sqm floor area should be considered for each ton of explosive. While stacking cases, each stack should not have more than 5 tiers and a working space of 1.22 m must be left between two stacks.

8.5.3 Where quantity of explosive exceeds 20 tonne, a separate building for storage of detonators must be built and for lesser quantity, detonators can be stored in an annexe, which is built as integral part of main building but has a substantial partition with an air space between them. As a rough guide, a double partition of 2 cm each with 45 cm air space between them will suffice the purpose of storing 10,000 detonators.

8.6 Blasting caps, electric blasting caps or primers shall not be stored in the same box or room with other explosives in big dumps. However, in small dumps sand bag revetments of appropriate thickness and height will be used to segregate different zones of explosives.

8.7 Explosives, fuse or fuse lighters shall not be stored in a damp or wet place or near oil, gasoline, or near radiators, steam pipes or other sources of heat.

8.8 Smoking and use of matches, naked lights and readily flammable articles or open fire/flame shall be prohibited with in the fenced area around it. Similarly, explosives should be kept away from electric contact, fuse boxes and switches.

8.9 An area upto a distance of not less than 50 m on all sides of a magazine shall be maintained free of all vegetation, debris and combustibles.

8.10 Metals, metallic objects and metal tools that are capable of producing sparks shall not be stored or used inside or in the immediate vicinity of the magazine.

8.11 Boxes of explosives shall not be thrown down or dragged along the floor and may be stacked on wooden trestles.

8.12 Package containing explosives shall not be allowed to remain in the sun.

8.13 Empty boxes, packing materials or any combustible material shall not be stored inside or in the vicinity of the magazine.

8.14 Adequate quantity of water and fire fighting equipment shall be provided near/in the magazine. Guards shall be properly trained in handling such equipment.

8.15 Signboards reading “DANGER-HIGH EXPLOSIVES”, “PROTECTED AREA” “NO SMOKING” etc. shall be prominently displayed in front of the magazine.

8.16 Well trained, preferably armed guards shall be posted to guard the magazine.

8.17 The following shall be hung up in the lobby of the magazine :

- A copy of explosive rules
- A statement showing the stock in the magazine
- Certificate showing the last date of testing of the lightening conductor

8.18 Magazine shoes, without nails, shall be kept at all times in the magazine and a wooden tub or cement trough, approximately 300 mm high and 450 mm in diameter, filled with water shall be fixed near the door of the magazine. Persons entering the magazine shall put on the magazine shoes provided for the purpose and be careful not to allow the magazine shoes to touch the ground outside clear floor. Persons with bare feet shall, before entering the magazine, dip their feet in water and then step direct from the tub on to the clean floor.

8.19 For continued blasting operations, the magazine shall be located at a safe distance near the work site and actual requirement of explosives for each blast may be drawn and transported to the site and left-over, if any, must be immediately returned to the magazine. Where the blasting operations extend to several scattered sites and/or one for a short duration, portable magazines shall be used. Each such magazine shall be located at a safe distance from the work site, enclosed in a fence and properly guarded.

9.0 SECONDARY BLASTING

Secondary blasting is used in the following cases-

- To break oversize boulders produced during the primary blast to suitable size.
- To break oversize boulders in landslides causing blockades.

There are two basic methods of secondary blasting viz. Pop Shooting and Plaster Shooting.

9.1 Pop Shooting

Pop shooting consists of drilling a hole just close to the centre of the boulder to be broken so that the charge is centrally situated and depth of hole is a little more than half to two thirds of thickness of the boulder. A hole is drilled either manually with the help of a cold chisel or with a hand drill or compressor drill as the situation may permit. The charge varies with the size of the boulder. The shots can be fired by safety fuse alone or in conjunction with detonating fuse. Pop shooting generally creates considerable scatter of rock and thus it is advisable to keep a minimum distance of nearly 200 m from the nearest structures. Machines/equipment etc. deployed near the site should be withdrawn to safer distances.

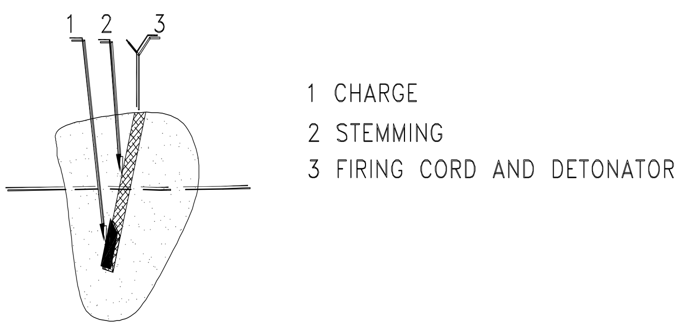


FIG. : POP SHOOTING

9.2 Plaster Shooting (mud capping)

Plaster shooting also known as 'Pressure Blasting' provides a ready means of breaking even large boulders in circumstances where drilling is difficult due to expediency, or due to non-availability of drilling tools/equipment at sites such as isolated landslides. A charge is laid on the surface of the boulder preferably in a cavity or in between the boulder and the ground and blasted after suitably mud capping the charge. Mud capping should be in good contact with the surface around the explosive charge.

In plaster shooting, the charge used is about four times that required for pop shooting, primarily depending upon the thickness of the boulder. Best results in plaster shooting are obtained when the rock is of a hard and brittle nature. Plaster shooting creates much noise which may lead to human psychological effect and may be heard within a periphery of 1 km. This method has to be adopted only in extreme urgency when pop shooting cannot be undertaken. Advantage of plaster shooting are that no drilling is required and little flyrock is produced.

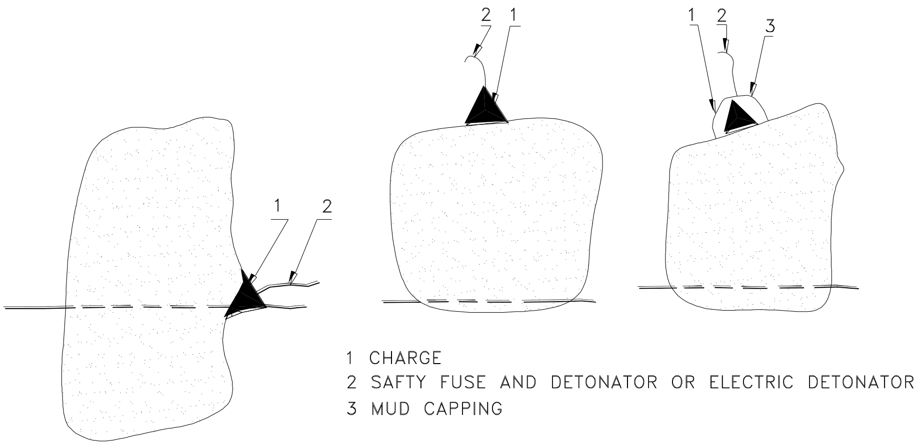


FIG. : PLASTER SHOOTING

10.0 Control Blasting

Uncontrolled blasting results in rough uneven contours, over break, over hangs, excessive shattering & extensive tension cracks. Blasting damage, therefore, can lead to significantly higher scaling, excavation, remedial treatment and maintenance cost. The results of blast shock wave and gases along faults, joints, bedding & discontinuities, although not readily apparent on the blast face, can lead to loosening of the rock.

In many excavations it is important to keep over break to a minimum. Apart from the cost of removing extra material which then has to be replaced, damage to rock forming the walls or floor may lower the bearing capacity and necessitate further excavation. Also smooth faces allow excavation closer to the pay line and are more stable.

Control blasting involves drilling closely spaced, carefully aligned drill holes, which are loaded with a light explosive charge, and detonated in a specified sequence with respect to the main blast. The explosive load is designed to generate a shock wave and gas pressure just sufficient to break the rock between the drill holes, but not to cause radial fracture in the rock behind the proposed slope face. It can be achieved by either using an explosive with a relatively low detonation velocity or by ensuring that there is an air gap between the explosive and wall of the drill hole.

Over break in a blast depends on the properties of the rock mass such as in situ dynamic tensile breaking strain & discontinuities and the type of explosive and its density. Over break also can be reduced by decoupling and/or decking back row charges. In both cases, the charge weight in the back row blast holes is lowered so as to reduce the strain wave energy, density and volume of explosion gases. However, less over break can only be guaranteed where the effective overburden on the back row decreases in proportion to the lower energy yield per blast hole. Charges are decoupled where the charge diameter is less than that of the blast hole. Minimization of over break can also be achieved by individual charges being initiated at one end (preferably the bottom) rather than at the centre.

Three methods of control blasting are commonly used. These methods involve the simultaneous detonation of a row of closely spaced, lightly charged holes, and are designed to create a clean separation surface between the rock to be blasted and the rock which is to remain. In each case, a row of blast holes are drilled along the line of the face. These methods differ in the blast sequence and the hole spacing.

10.1 Line drilling

Line drilling consists of accurately drilling alternate small-diameter holes between the pattern blast holes forming the edge of the excavation. The quantity of explosives placed in each line hole is significantly smaller and, indeed, if these holes are closely spaced, from 150 to 250 mm, then explosive may be placed only in every second or third hole. The closeness of the holes depends upon the type of rock being excavated and on the payline. These holes are timed to fire ahead, with or after the nearest normally charged holes of the blasting pattern. The time of firing is largely dependent on the character of the rock involved. Line drilling is not always successful in preventing over break although it helps to reduce it. Generally, line holes in sedimentary and some metamorphic rocks are not as effective as in igneous rocks.

10.2 Cushion Blasting

Cushion blasting is a type of line drilling where large holes, about 170 mm diameter, are located between the line holes to act as guides to the crack direction. Frequently one large hole to three small holes are used with either all three small holes or only the large central one being charged. The holes are loaded with light charges at intervals separated by stemming. The trimming holes are fired after the main blast. Cushion blasting often gives better results than line drilling.

10.3 Pre - split Blasting

Pre-splitting can be defined as the establishment of a free surface or a shear plane in rock by the controlled usage of explosive in approximately aligned and spaced drill holes.

In this technique, a row of closely spaced and usually small diameter holes is drilled along the line of the final face. The spacing of the holes is governed by the type of rock and the diameter of the holes. The diameter of the holes is smaller than that of the main blast holes in order to avoid the extra radial cracking of the walls. These holes are lightly charged and the charge is decoupled from the rock by leaving an air space between the charge and the walls of the blast hole. This is usually achieved by placing the charge in a plastic tube of smaller diameter than the hole and centering this tube in the blasthole with some form of spacer.

The row is fired **before** the main charge and the reinforcing effect of the closely spaced holes together with the very large burden results in the formation of a clean fracture running from one hole to the next.

Pre split holes should normally be drilled at approximately 70° to the vertical but, if the available blast hole rigs can only drill vertical holes, the holes should be drilled along the line of the toe of the proposed bench.

This technique is mainly utilized in hard rocks. Pre-split blasting is not usually successful in well jointed hard rocks, particularly where the joints are open and are inclined to the pre-split line. These open joints allow the explosion gases to vent and fracturing follows the joints rather than the intended pre-split line.

Pre-spliting also requires accurate drilling. Air decking is sometimes used, that is, the charges are separated by spacers instead of stemming. The air space provides effective decoupling and damps the shock wave transmitted to the rock mass. Nevertheless this is time consuming and can prove troublesome. In weak rock alternate advance pre-split holes may be left uncharged. The uncharged holes form relief holes which guide the shear fracture along the required plane of separation. Maximum effectiveness is achieved when the pre-split holes are fired simultaneously. A trial should be made to determine whether the site conditions require modifications in the hole spacing or charging prior to extensive pre-splitting taking place.

B. MAINTENANCE ASPECTS

1.0 Remedial work should be planned by engineering staff with experience, using consulting advice where needed. From an analysis of site conditions, the most suitable treatment of a dangerous slope can be chosen. Methods of treatment should be considered in the following priority-

Priority 1- Stabilization measures, i.e. preventing rocks or slope material from moving out of place unexpectedly
Priority 2- Protection measures, i.e. keeping rocks which do move out of place from reaching the track
Priority 3- Warning traffic when rocks or slope material arrive in the vicinity of track

Stabilization and protection measures offer a positive solution to the problem and warning methods by themselves have no effect on the causes of danger.

1.1 Landslides occur most frequently at time of high ground water or heavy rainfall. They seldom occur without advance cracking of the ground or other signs. Therefore, constant vigil and watch is required and suitable speed restriction may be imposed during periods of intense rainfall, if required.

2.0 Though accurate prediction of rock falls is not possible, however, the risk of rock falls at particular locations should be assessed in a general way by experienced staff. A regular inspection of these locations should be made for appraisal of hazards and action & priorities should be decided accordingly.

3.0 Earth slopes are subjected to the continuing effects of gravity and running water. As a result, periodic maintenance is required to clean ditches, drains, fill gullies and prevent erosion by rain.

3.1 An earth slope either above or below track which has failed so as to involve the safety of running traffic must be repaired immediately. However, repairs should tie in with long term stability requirements and should be carried out with the basic principles of slope stability in mind. Neglect of this will lead to recurring instability or added costs in subsequent restoration work.

3.2 In an earth slope, there are forces tending to cause sliding and forces resisting sliding. Forces causing sliding are gravity and water. The resisting forces are made up of the strength of the soil along the sliding plane in the slope, and weight of earth tending to be displaced at the toe of the slope. Slides occur when sliding forces increase or resisting forces decrease enough to cause movement. The effectiveness of any form of slope stabilization will depend on how it affects those two forces.

3.3 Planning for stabilization of an unstable slope should always start with a thorough inspection of the slope, if possible with an experienced geo-technical engineer, to find out what is happening. The inspection should extend up and down hill from the track for a distance well beyond the obvious signs of instability. A local failure will sometimes be part of a much larger movement going on in the general area.

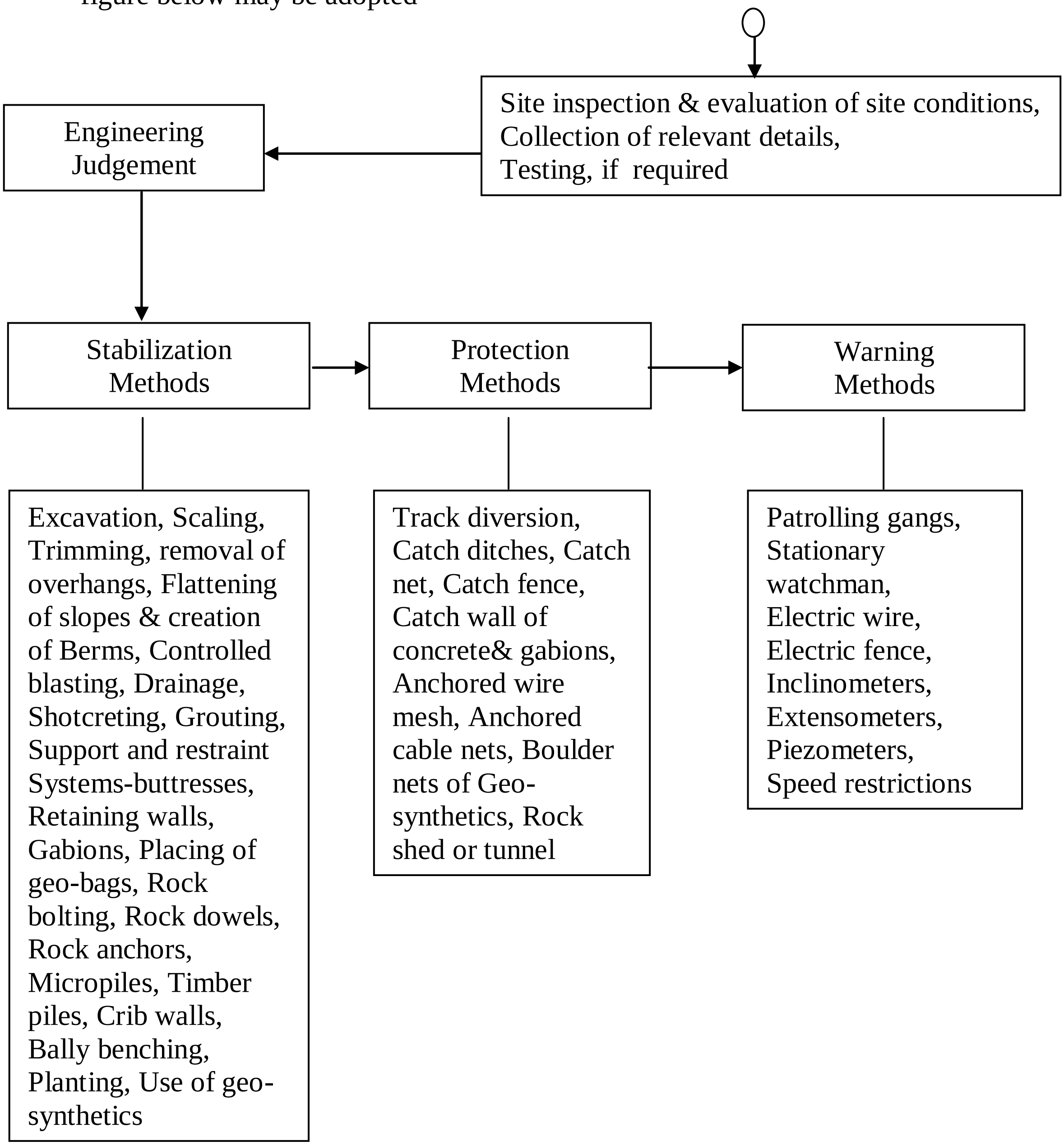
3.4 During inspection, observations should be made of

- Any recent excavation or filling
- Crack or bulges in the slope,
- Water seeping into or out of the ground wet areas,
- Direction, extent and depth of the movement, judging by the displacement of ground cracks, tracks, fence lines or leaning trees.

These observations should be supplemented by an analysis of the slope stability, based on drilling, soil sampling and testing whenever warranted by the importance of the problem and the time available.

3.5 Methods of restoring slope stability are chosen on the basis of site observations and analyses made and the suitability, feasibility and economies of the various alternatives. It is sometimes possible to gain time to implement these measures by temporarily moving the track away from the instable area.

4.0 Depending upon site condition and evaluation thereof, approach shown in the figure below may be adopted-



5.0 CRITERIA FOR VULNERABILITY

Vulnerability of cuttings is to be established after evaluating the risk potential that the cutting poses to the traffic and workmen from critical study of various factors involved, a few of which are enumerated below-

- Depth of cutting/slope height
- Steepness of slope
- Existence of loosely held rock chunks/boulders, open joints in strata over the slope
- Geological condition of strata:
  - Closely jointed/fractured nature of rock
  - Unfavourable joint orientation
  - Degree of weathering in rock
  - Discontinuities
  - Rock friction
- Heavy seepage of under ground water at soil-rock interface during monsoon period
- Negative slope/undermining in middle & bottom of cutting
- Overhangs in top section of cutting.
- Existence of soft zones in middle and bottom of cutting
- Large catchment area but non-existence of catchwater drain
- History of soil slips/rock slips/rock falls
- Blocksize
- Quantity of rock fall
- Ponding of water on top of cutting
- Presence of tension cracks
- Catch ditch effectiveness
- Erosion of slopes
- Climate, rail fall, presence of water on slope
- Cuttings on approaches of tunnel & bridges
- Slope length, average daily traffic & sectional speed: this will indicate the time for which a train will be present in the cutting zone
- Sight distance available to train driver

5.1 Each Divisional Engineer/Sr. Divisional Engineer should identify the vulnerable cuttings in the various sections.

6.0 TROLLY REFUGES

Trolley refuges of appropriate dimensions are to be provided in cuttings at suitable intervals. However, the maximum distance apart of trolly refuges shall not exceed 100 m. For easy identification of the location of trolly refuges in cuttings, a distinguishing mark such as a rail post, painted with luminous paint with mark ‘R’ may be erected by the side of the trolly refuge.

7.0 Inspection Steps

In deep cuttings, inspection steps should be conveniently located to provide access to top portion of the slope for facilitating inspection. Provision of hand railing may also be considered depending on the site specific requirement.

8.0 RECORD OF MISHAPS

Good records are the basis for good planning and priorities. Database of all the events in cuttings such as landslides/rock falls which are capable of disrupting traffic or causing injury to the officials, or traveling public should be maintained in the **permanent way inspector's section register** as per the proforma given at Annexure -IV. The database may also be maintained in planning cell of division as well as HQ office. The database will help in formulating measures to prevent recurrence of such events in future.

9.0 INVOLVEMENT OF EXPERTS AND PRIVATE AGENCIES

Expert Government bodies like GSI, CRRI or geological/geotechnical experts of repute may be consulted for specific problems under advice to RDSO.

10.0 INSPECTION OF CUTTINGS

10.1 Cutting Register

A register for these cutting should be maintained in the proforma given in Annexure-V. Separate page will be maintained for each cutting.

10.2 Schedule of Inspection

10.2.1 Immediately after the monsoon, the PWI should inspect each cutting and record his observation in the register which should be sent to the AEN for his examination well before the next monsoon to enable for planning of remedial measures that he may like to take in the intervening period.

10.2.2 Each cutting should be inspected before the onset of rains by the AEN concerned and he should record his remarks in the register which should then be sent to the PWI for taking appropriate action. Action taken by the PWI should be recorded in the register and the same returned to the AEN for his perusal before the onset of the monsoon. Date by which these registers should be returned to the AEN for his perusal to enable that adequate action has been taken should also be specified by the Sr. DEN/Co-ordination depending upon the time when the monsoon starts in a particular section.

10.2.3 During monsoon, frequent inspection of vulnerable cuttings may be carried out as required keeping in view the past history and the vulnerability of the cutting.

10.2.4 Each Divisional Engineer/Sr. Divisional Engineer should inspect the cuttings referred to him by AEN and by territorial HOD if referred by Divisional Engineer/Sr. Divisional Engineer.

10.2.5 During spell of heavy rains, the AEN & PWI should inspect by trolley, foot-plate of the engine or other means the cutting and allied works as frequently as possible.

10.3 <u>Points to be noted during inspection of cuttings</u>

During inspection, points given in checklist of Annexure-VI should be seen in detail.

10.4 <u>Action to be taken in the case of boulder drops</u>

10.4.1 In case of boulder drops, the boulder may be removed by jacking. If the boulder cannot be moved by jacks or levers, hydraulic splitting or blasting will be necessary.

10.4.2 Inspectors who will handle blasting equipments should be conversant with the methods of blasting and should be familiar with all safety precautions to be observed for the custody and use of explosives.

10.4.3 The following equipment should be kept at the Head quarters of each inspector in whose section such vulnerable cutting exist :

- Jacks in good working condition,
- Hydraulic splitters
- Jumping steel bars 1'' dia and 5'long,
- Charging rods and
- Suitable stock of explosives, fuses and detonators at specified places.

10.5 <u>Action to be taken after inspection of cutting</u>

After inspection of cutting, deficiencies noticed during the inspection should be removed and short term as well as long term action plan for protection & stabilization of cutting should be framed as per para 4.0 above and executed.

10.6 <u>Guarding of Vulnerable Cuttings</u>

Stationary watchmen should be posted round the clock at nominated vulnerable cuttings during the monsoon period in accordance with para 1014 of Indian Railway Permanent Way Manual 1986.

CHAPTER IX

CASE STUDIES

A few case studies highlighting measures adopted for rockfall prevention and slope stabilization on Konkan Railway are detailed below:

1.0 UNDERI CUTTING

Section	Karanjadi - Diwankhauti
Chainage	km 67/500 – 67/700 – on both sides, depth – upto 15 m.
Type and nature:	• mainly blocky & jointed basalt rock, cutting with sub-vertical slopes, • sub-vertical & sub-horizontal joints prominent in rock, • sub-vertical joints sub-parallel to track
History of Problem	boulder fall/rock slip & derailment on three occasions on 07.10.99, 19.08.2000, and 30.08.2002.
Remedial Measures Adopted	• extensive loose scaling of rock slope • rockbolting to pin sub-vertical joints • high strength steel boulder netting over rock slopes
Sketch	cross section showing rock slip @ km 67/622 (see figure 1)
Effect of Measures	• initially only rockbolting over rock slope done, • after the 2nd derailment, HSBN was provided as rock fall protection measure, • no unusual noticed thereafter.

Specification of high strength steel wire rope net as adopted by KRCL are as follows-

- *Description* — galvanised steel (9mm)
- *Mesh opening size* — 450 mm x 600 mm
- *Size of the rope* — 9 mm nominal
- Tensile strength in both *the directions*
  - *a) Rope* — 4000 kg
  - *b) Rope net: Vertical, Tv* — 8.5 ton/m
    Horizontal, Th — 6.7 ton/m
- *Breaking strength, punching shear strength and other tests* — as given below
- *Material of the rope* — galvanised steel
- *Construction of the net* — joints tucked in one direction and clamped, ends of horizontal cords spliced or mechanically clamped.

- *Structure of the net* — rolls of 5 m width with a tolerance of +5%, the nets can be supplied in lengths of 15, 20, 25, 30, and 35 m length, the net can be supplied with hangs of 6 m, 8 m, 10 m & 12 m length as required.

Breaking strength test of ropes and nets

- The breaking strength of the rope/the net shall be tested by applying a tensile load at a constant rate of loading.
- The minimum length of the rope / net used for this test shall be 1000 mm.
- The ends of the rope/net shall be looped for applying the tensile load. Any gripping device, which is likely to damage the rope/net, shall not be used.
- The width of the rope/net sample to be tested shall be 450 mm (i.e. 2 chords) for vertical direction. Rope net in horizontal direction shall not be tested and the results shall be calculated with the basic data obtained from vertical direction rope net test.
- The elongation of the rope/net shall be measured at load increments of 1000 kg.
- A stress strain curve shall be plotted for each test. Breaking load/m = Breaking load to test piece/number of longitudinal ropes in the test piece/mesh opening size in m.
- Strain at failure and breaking load shall be recorded for each test.
- The breaking strength of the rope shall be maximum load reached during the test.
- The breaking strength of the net shall be that load at which breakage in any one rope takes place.

Punching shear test on the net

- For the purpose of conducting the punching shear test, a test frame of 1000 mm by 1000 mm size shall be fabricated.
- The sample of the net to be tested shall be of size 2000 mm x 2000 mm.
- The net shall be stretched over the edges of the frame and tightly stretched and secured. The stretch in the net shall be such that the sag of the net at the midpoint shall not be more than 50 mm.
- A punching force perpendicular to the plane of the net shall be applied. The application of the load shall be through a bearing plate of size 800 mm x 800 mm.
- Deflection of the midpoint of the net shall be recorded at every load increment of 500 kg upto failure.
- The maximum load applied on the net shall be upto 90% of the breaking strength of the rope net i.e. upto 7.6MT.

Other tests

Breaking strength of spiral lock springs, leaf opening test of clamps for load specific direction.

2.0 KOTWALI 'NORTH' APPROACH CUTTING

Section	Anjani - Chiplun
Chainage	km117/180 - 117/220, depth - upto 27 m.
Type & Nature	• basalt rock-laterite soil cutting, mainly on east, • top 8 m to 10 m - Laterite soil with steep slope, • middle section- in highly weathered basalt, . • bottom 8 m to 10 m section – in jointed basalt.
History of Problem	major soil slip on 17th July, 99 from eastern cutting slope - with approx.1500 cum. of soil slip coming down the slope.
Remedial Measures Adopted	• removal of loose/slipped soil earth • flattening of top soil slope on east • creation of berm at soil-rock interface • providing & lining of catchwater drain
Sketch	sketch plan & cross section showing major soil slip on 17th July, 1999 (see figure 2 & 3)
Effect of Measures	stable after the completion of safety measures

3.0 AGAVE CUTTING

Section	Savarda - Arawali Road
Chainage	km 148/800 - 149/080, depth - upto 23 m on both sides.
Type & Nature	• basalt rock - laterite soil cutting • top 4 m to 6 m section - in laterite soil, • middle section - weathered basalt & soft red bole zone(2 m thick), • bottom 8 m to 10 m section - foliated/vesicular • basalt with sub-vertical slope, • seepage from red bole zone, • undermining from soft red bole zone, • overhang in top, weathered rock above red bole zone & spalling of rock strata in middle & bottom section noticed, • cutting also close to track. ,
History of Problem	small soil slip/soil erosion , boulder falls reported in monsoon 1998, 1999, 2000 & 2002.
Remedial Measures Adopted	• loose scaling of rock slopes, • provision & lining of catchwater drain, • 100 mm thick shotcrete with wiremesh over soft red bole zone & weathered rock strata, • 50 mm thick shotcrete over rock slopes, • HSBN over the rock slope being provided.
Sketch	cross section showing small soil/rock slip on 08.06.2000 from east hill slope @ km148/915 (see figure 4).

Effect of Measures	no unusual reported after providing above mentioned safety measures

Specifications for shotcreting as adopted by KRCL are as follows-

- The shotcrete inside cutting shall be of concrete of grade M25 with 10 mm maximum nominal size of aggregate. The cement, sand, aggregate for shotcretting shall be conforming to IS 8112-1976 and IS 383-1970.
- Before shotcreting, rock surface shall be cleaned with water and compressed air.
- The cement used for shotcreting should conform to IS 8112-1976 or IS10262-1982. The cement brought to site shall be approved by the engineer in charge or his representative before starting the work. Cement bags which are older than three months from the date of manufacturing shall be summarily rejected. Contractor shall submit a test certificate from the manufacturer regarding the quality of cement.
- The shotcreting operation shall be continuously supervised by the contractor's supervisor who checks the materials, delivery equipments, application of material, curing etc. Layer of shotcrete shall be systematically sounded with a hammer to check the dummy area.
- Weep holes of maximum 38 mm dia. and at such locations/spacing as directed by engineer-in-charge shall be provided in shotcrete.
- All costs of developing shotcrete mixes shall be at the expense of contractor.
- Nozzlemen shall have had previous experience in the application of the shotcrete on at least two projects.
- The thickness of shotcreting shall be measured by drilling a 25 mm dia hole at random, at the rate of one hole per 100 sq.m. to ensure that the shotcrete has been done to correct thickness, nails will be driven in the rockface @ one nail per 5 sq.m. The head of the nail should project by the desired thickness of the concrete. The shotcrete will be started only after a representative of KRCL certifies the provision of nails. After completion of work, no nail should be visible on the face of the shotcrete.

4.0 KHERSHET SOIL CUTTING

Section	Savarda - Arawali Road
Chainage	km153/465 - 153/535, depth - upto 25 m.
Type & Nature	laterite soil cutting – mainly on west, steep soil slope, high depth & large catchment area.
History of Problem	• major soil slip on 9th July, 2000 - from western hill slope, subsequent to continuous & heavy rain fall, • huge quantity of soil earth slipped down the slope, with slip circle, just 5 m to 6 m above the rail level i.e. soil-rock interface, • the slip resulted in suspension of rail traffic for two weeks
Remedial Measures Adopted	• earthwork done to flatten soil slope in1:1 & provide berms at various levels.

	• catchwater drain provided & lined. • geomatting was done over soil slope. • RCC toe wall was provided
Sketch	sketch plan & cross section showing soil slip from western cutting slope (see figure 5 & 6)
Effect of Measures	stable after completion of all safety works.

5.0 GOLAVALI SOIL CUTTING

Section	Arawali Road-Sangmeshwar
Chainage	km166/200 - 166/450, depth - upto 30 m on east & 12 m on west.
Type & Nature	• laterite soil cutting with high depth upto 30 m on east & 12 m on west, • bottom 10 m to 12 m - hard laterite/soft murum rock, • middle & top section - loosely consolidated laterite soil mixed with subrounded boulders.
History of Problem	• soil slip from eastern hill reported during construction stage of 94 - 96, • major soil slip in July, 97 from eastern hill, distinct soil slip with subsidence along slip circle, numerous cracks & disturbance of soil slope noticed upto 10m above rail level i.e. above hard laterite.
Remedial Measures Adopted	• earthwork done to flatten in 1:1 slope & berms at various levels on east, • providing & lining of catchwater drain– on top & on berms, • geomatting over eastern soil slope. • providing micropiles in two rows on 1st berm length, with pile cap & RCC wall over it to prevent movement of entire soil mass behind.
Sketch	sketch plan & cross section showing soil slip from eastern hill slope (see figure 7 & 8).
Effect of Measures	no unusual reported after completion of safety works.

6.0 KHEDSHI CUTTING

Section	Bhoke - Ratnagiri
Chainage	km199/400 - 199/620, depth - upto 30 m.
Type & Nature	• basalt rock - laterite soil cutting on both sides, • depth upto 30 m, • bottom 8 m to 10 m in weathered / jointed basalt with sub-vertical slope, overhang, etc. • top 18 m to 20 m in laterite soil having steep slope

History of Problem	• boulder fall in monsoon 99. • boulder fall & derailment of loco on 22.03.2000.
Remedial Measures Adopted :-	• extensive loose scaling of rock slope, • overhangs were removed controlled blasting. • 100 mm thick shotcrete with wire mesh provided over rock slopes on both sides, • spot rockbolting done to pin unfavourable joints, • damaged UCR walls replaced with RCC walls, • concrete jacketing done over UCR walls in some length, • earthwork done to flatten slope & create berm on top soil strata on both sides, • geomatting done on soil slopes to prevent erosion, • lined catchwater drain provided.
Sketch	cross section showing boulder fall from eastern hill slope (see figure 9).
Effect of Measures	nothing unusual reported after completion of above works

Specification of rock bolting as adopted by KRCL are as follows-

- Material - TOR steel of 25 mm diameter
- Length - The length of the rock bolt shall be 3.0 m.
- The rock bolt at one end shall be threaded upto 15 cm length while the other end shall be pointed.
- The nut of rock bolt shall be compatible with the bolt and hexagonal in shape.
- The M.S bearing plate will be of 150 mm x 150 mm size and 8.0 mm thick. The hole in the bearing plate should be of 30 mm dia.
- Plain/beveral washers compatible with rock bolt shall be used for fitting with rock surface.
- Full length of the rock bolt shall be anchored by cement/grout capsules (5 capsules shall be consumed per rock bolt).
- The threads on bolts and nuts shall be lubricated and suitably protected.
- The rock bolt will be commercially straight, sound and free from surface defects.
- The rock bolts in arch and walls of the tunnel shall be installed in a radial manner unless stated otherwise. The bolts will be normally installed in a staggered manner as per instructions of engineer-in-charge.
- The rock bolts will be properly tightened/tensioned by torque wrench/impact wrench upto a minimum load of 15 T immediately after setting of cement capsule/grout.
- The entire rock bolting work will be divided into lots of 200 bolts for purpose of testing for acceptance/rejection. The tested bolts will be identified with paints or tags. The rock bolt should withstand a pull out strength of 15 tons (force). 5% of these 200 bolts will be selected randomly for pull out test and maximum 10% of failure will be permitted. If the failure is less than or equal to 10% of the tested bolts, the entire lot of 200 bolts will be accepted. If failure is more than 10% of the tested bolts, another 5% of rock bolts will be

selected randomly in the same lot for testing. If the failure is less than or equal to 10%, the entire lot will be accepted. If failure is more than 10%, the entire lot of 200 bolts will be rejected and the contractor has to put extra 200 rock bolts at his own cost with all materials, labour, etc. If during the pull out test, if the bolt comes out along with the rock mass i.e. the failure at the level of rock, then the bolt will not be deemed to have failed. The decision of engineer-in-charge or his representative regarding this is final and binding on contractor. Regardless of final acceptance of the lot, the contractor will have to install rock bolts to replace all those bolts which have failed.

- The contractor shall arrange to procure torque wrench and pull out test equipment for carrying out necessary tightening and pull out tests.

7.0 KONDAVI SOIL CUTTING

Section	Ratnagiri - Nivsar
Chainage	km 218/100 - 218/300,
Type & Nature	• laterite soil-high depth on east, loosely consolidated, with small & big size subrounded boulders mixed in soil strata, • tendency of erosion/slip, heaving of track formation noticed during monsoon.
History of Problem	• major soil slip from eastern cutting slope with downward movement of entire hill was noticed, in the month of June & July 1997, • the slip circle was below the formation, resulting in heaving of formation & shifting of rail track, • the traffic remained suspended for about one week due to this slip.
Remedial Measures Adopted	• earthwork done to flatten slope on east on very large scale. • micropiling in two rows with pile cap provided at toe of the cutting as slip circle was below formation, this helped in stopping downward movement of entire hill. • catchwater drain provided.
Sketch	sketch plan & cross section showing soil slip & measures taken (see figure 10, 11 & 12).
Effect of Measures	stable after completion of all safety works.

8.0 MIDC-KUDAL SOIL CUTTING

Section	Kudal - Sawantwadi
Chainage	km 344/800 - 344/950, depth - upto 20 m on east & 10 m on west.
Type & Nature	• laterite soil - clay cutting with high depth upto 20 m on east & 10 m on west, • top 2 m to 4 m - in hard laterite – cracks, overhangs

	observed – mainly on east, • bottom section - in very soft, clayey soil strata with tendency of erosion/undermining in every monsoon.
History of Problem	• major soil slip due to undermining/ erosion of middle & bottom soft clay strata on east@ km 344/800-930, in July'97, • this resulted in detachment of top laterite soil boulders& pushing down of soft loosely consolidated clay strata &damaging the toe walls, • the slip circle was found to be below the formation with heaving of formation& shifting of track noticed.
Remedial Measures Adopted	• earthwork done to flatten soil slope, • laterite overhangs removed, • micropiling done with pile cap in rows at toe & 2 rows at middle level to arrest lateral & downward movement of disturbed soil mass, • gabions provided over pile cap, • stone pitching done in undermined locations & over soil slope between piles, • side drains deepened & lined.
Sketch	sketch plan & cross section showing soil collapse (see figure 13).
Effect of Measures	stable after completion of all safety works

Specification of gabions as adopted by KRCL are as follows-

- Gabions are manufactured from Zinc coated, double twisted, hexagonal woven, wire mesh, conforming to ASTM-856 standards. Dia of wire is 2.70 mm.
- Dia of selvedge wire-3.40 mm
- Dia of lacing wire - 2.20 mm
- Minimum Zinc coating - 240gm/sq.mt (BS-443:82)
- Minimum Tensile strength – 350 Mpa (BS- 1052:80)
- Mesh size 100 mm x 120 mm
- Minimum elongation of 10% at breaking load
- Non-woven eotextile filter to be provided behind gabion

9.0 WAMNE CUTTING

Section	Veer - Karanjadi
Chainage	km 54/170 - 54/850 depth - upto 25 m on east.
Type & Nature	basalt rock - laterite soil cutting top 6 m to 8 m - in laterite soil, steep slope middle section - in weathered basalt, steep slope bottom 10 m depth - jointed basalt, subvertical slope open joints/cracks seen in rock.
History of Problem	soil erosion } soil slip } in monsoon of 96, 97, 98 &

	boulder fall } 2000
Remedial Measures Adopted	• loose scaling of rock slope, • flattening of top soil slope in 1:1 slope, • 5 m wide berm provided at soil - weathered rock interface, • 5 m wide berm provided at weathered rock - rock interface, • catch water drains provided on top and on berms & lined.
Sketch	cross section of cutting @ km 54/800 (see figure 14).
Effect of Measures	stable after safety works

CASE STUDY –1

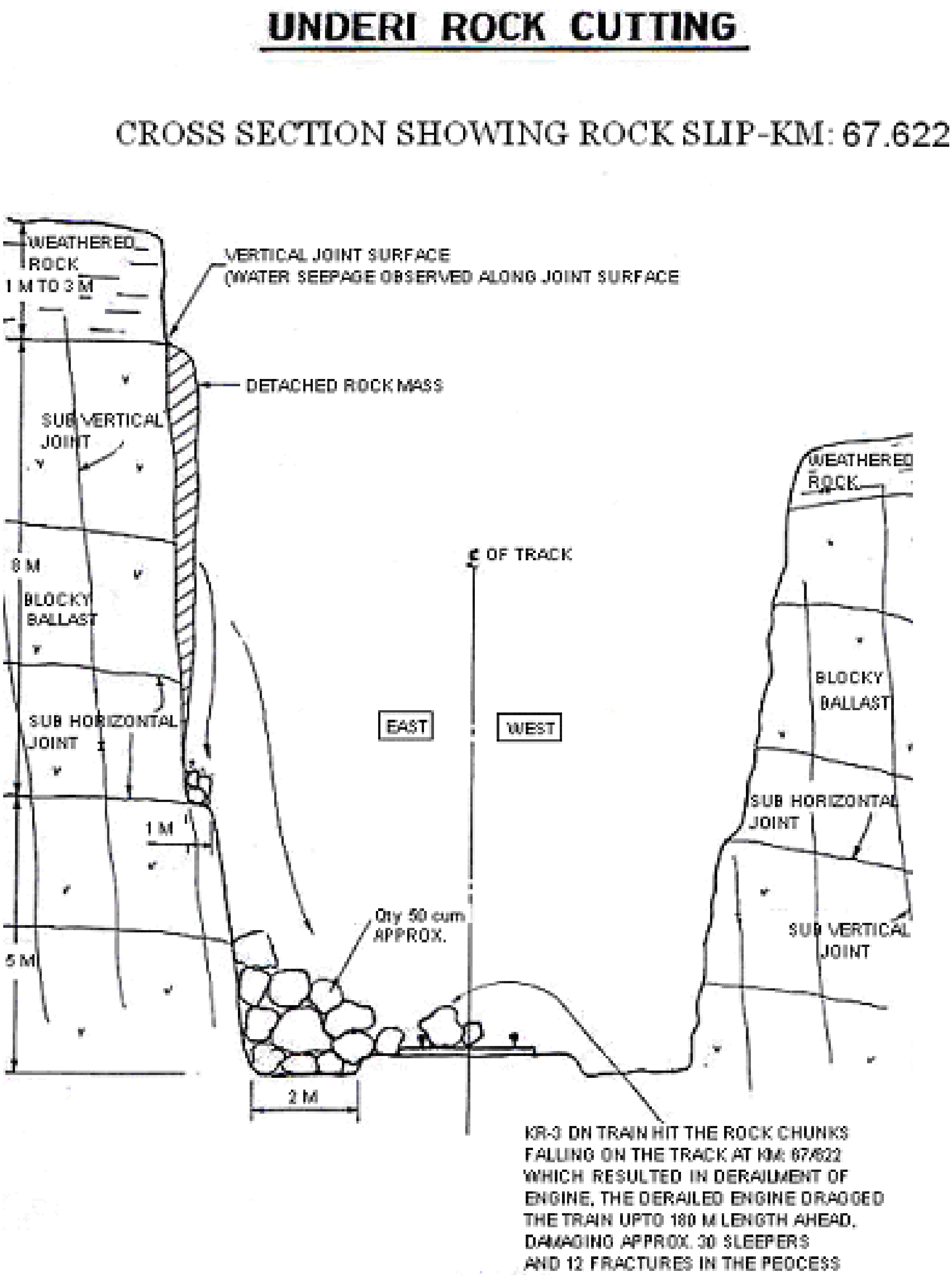


Fig. 1

CASE STUDY – 2

SKETCH PLAN SHOWING SOIL SLIP AT KM. 117/190-220 FROM EASTERN HILL SLOPE ON 17-07-1999

(KOTWALI 'N' APP. CUTTING)

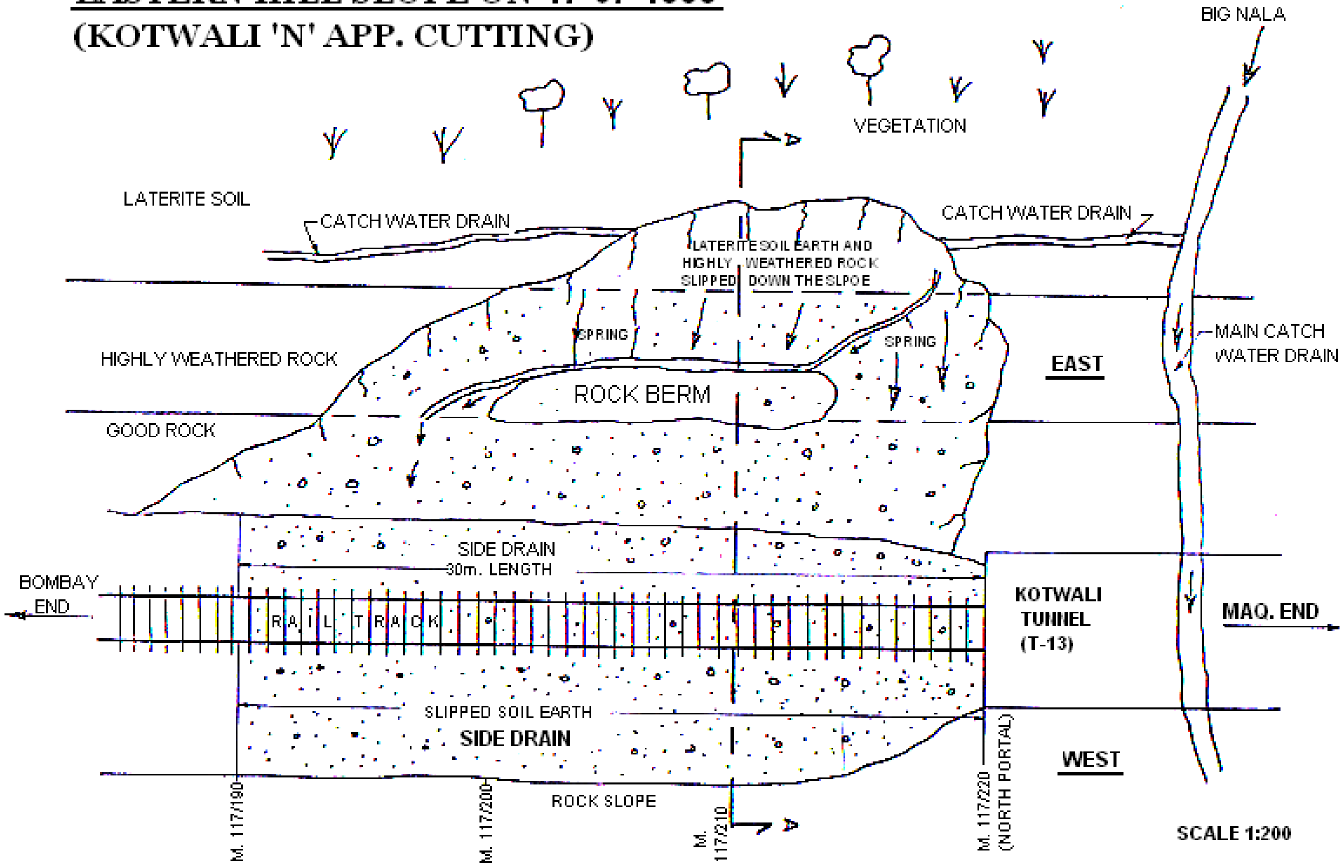
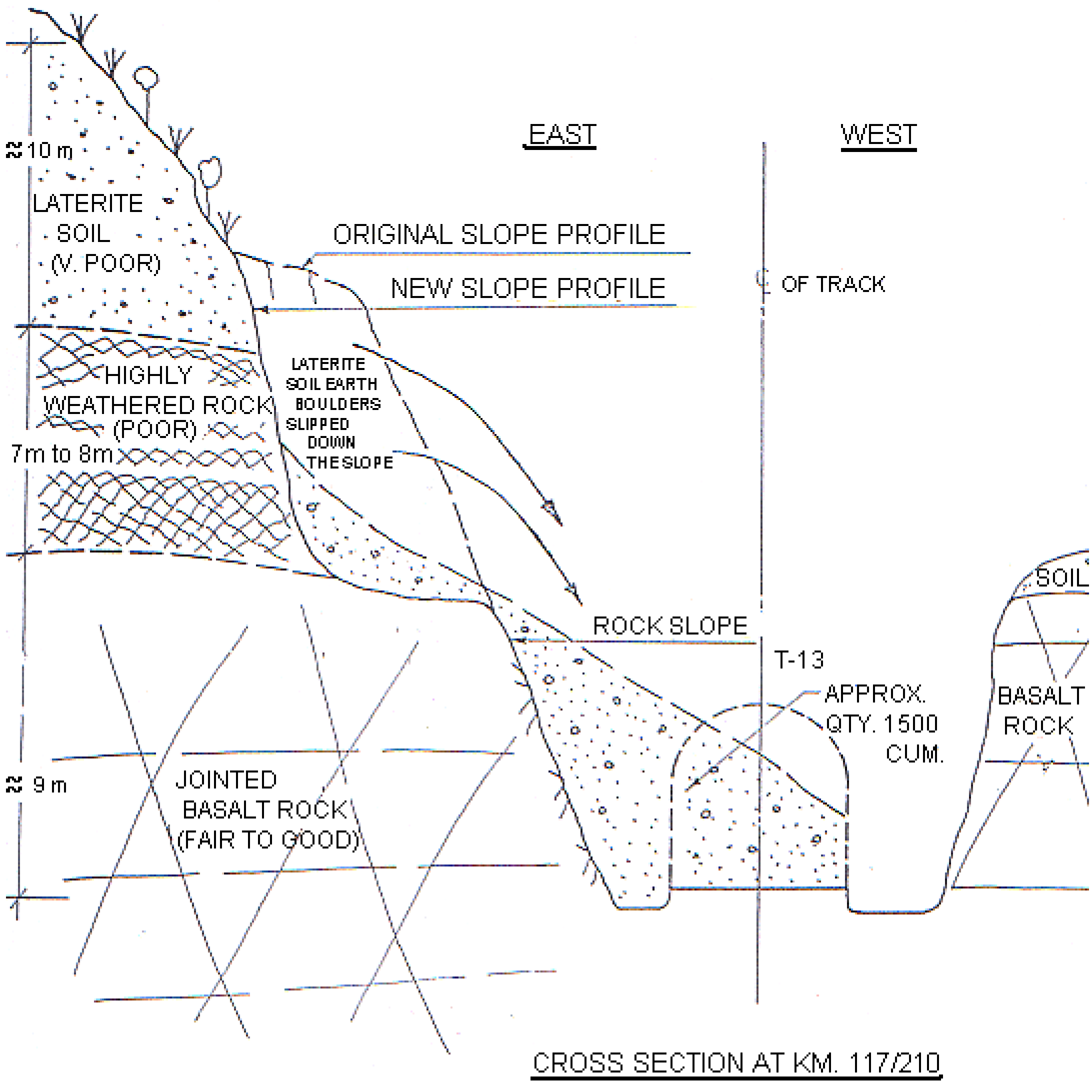


Fig. 2

CASE STUDY – 2

CROSS SECTION AT KM. 117/210 SHOWING SOIL SLIP FROM EASTERN HILL SLOPE ON 17-10-1999. (KOTAWALI 'N' APP. CUTTING)



SCALE 1:200

Fig. 3

CASE STUDY – 3

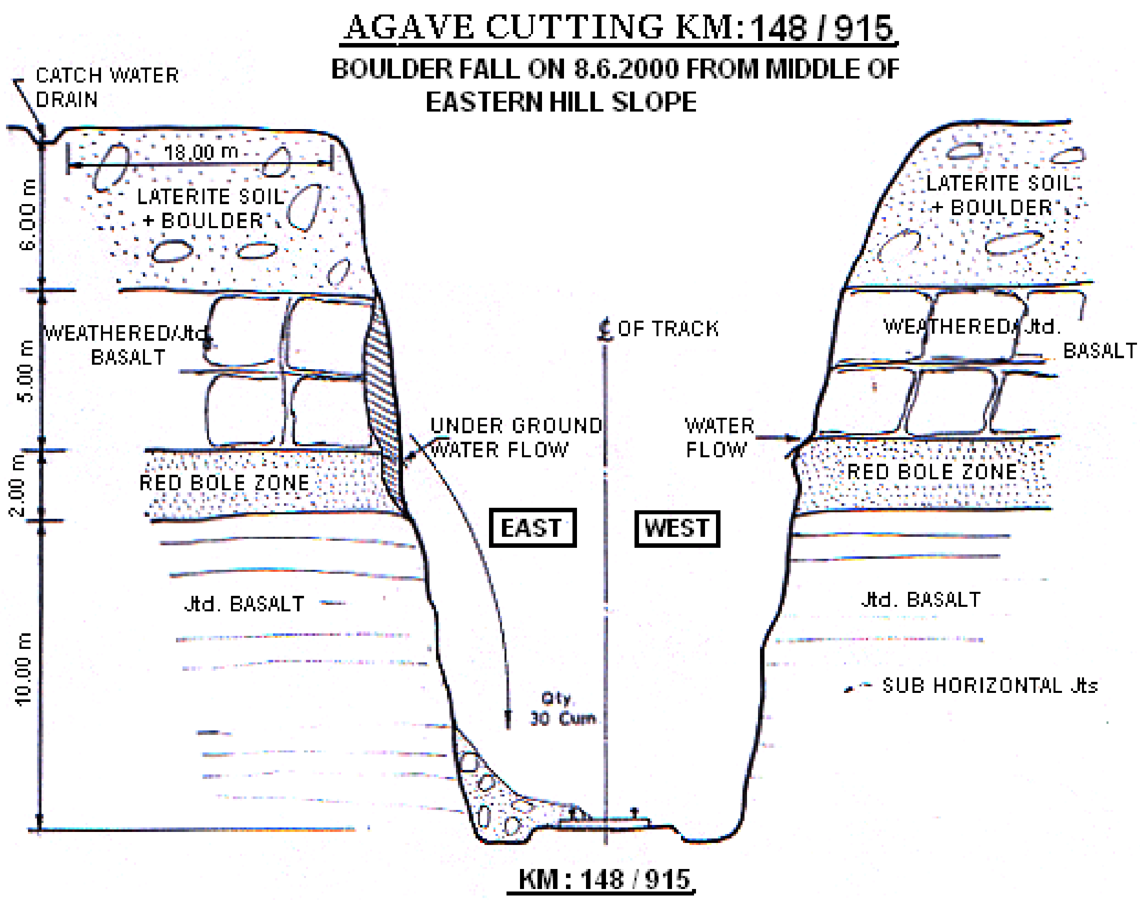


Fig. 4

CASE STUDY – 4

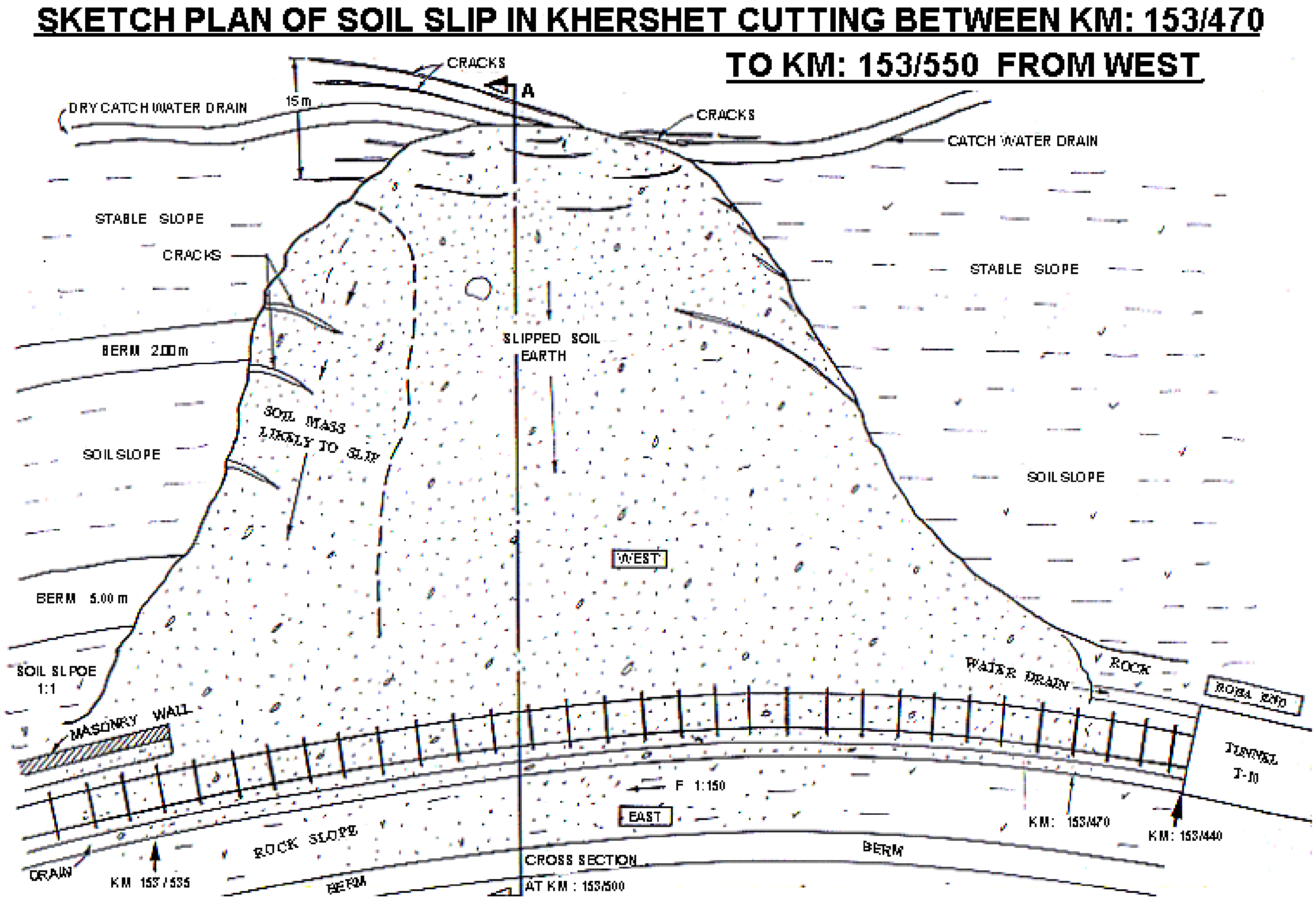


Fig. 5

CASE STUDY – 4

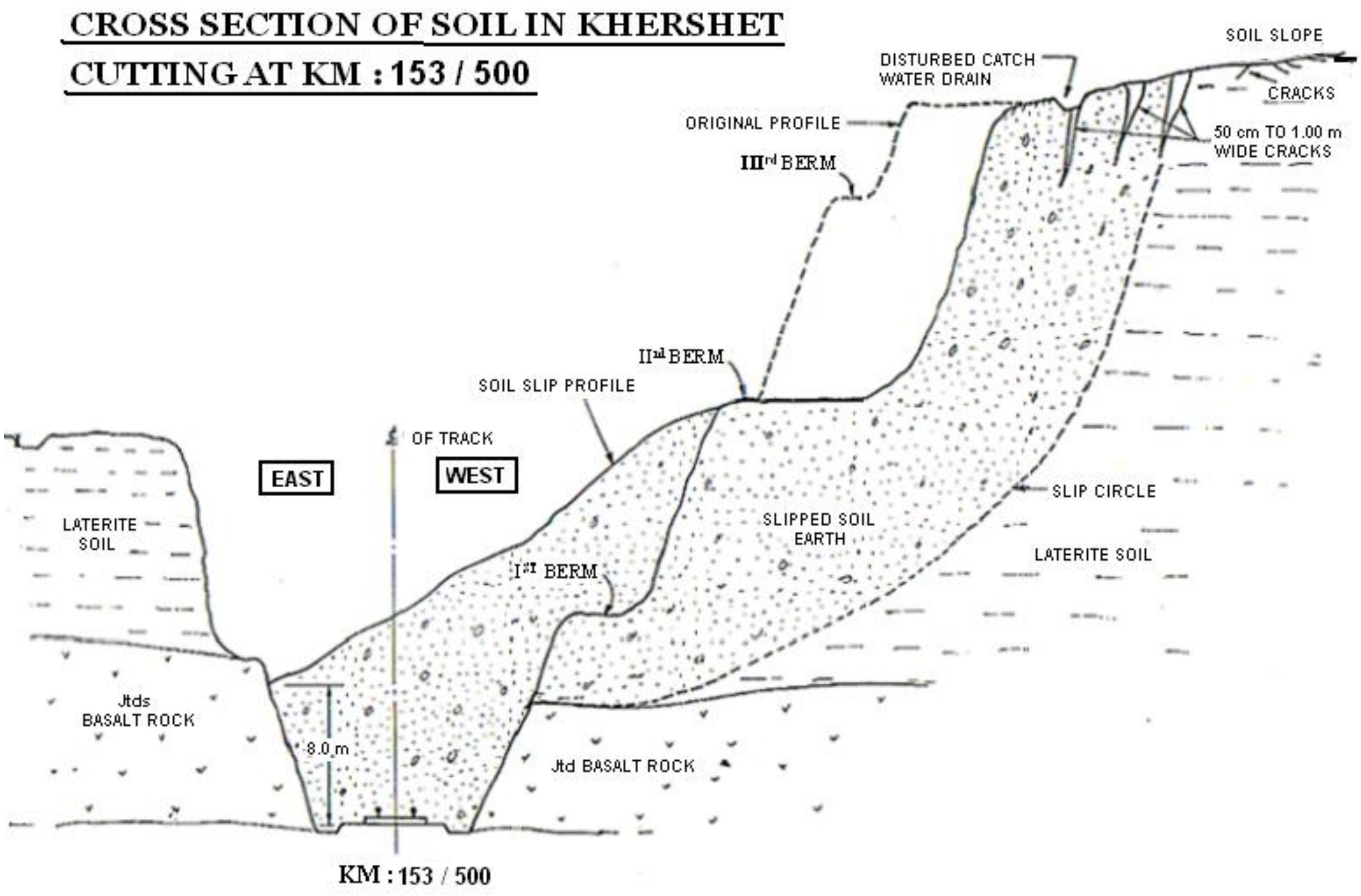
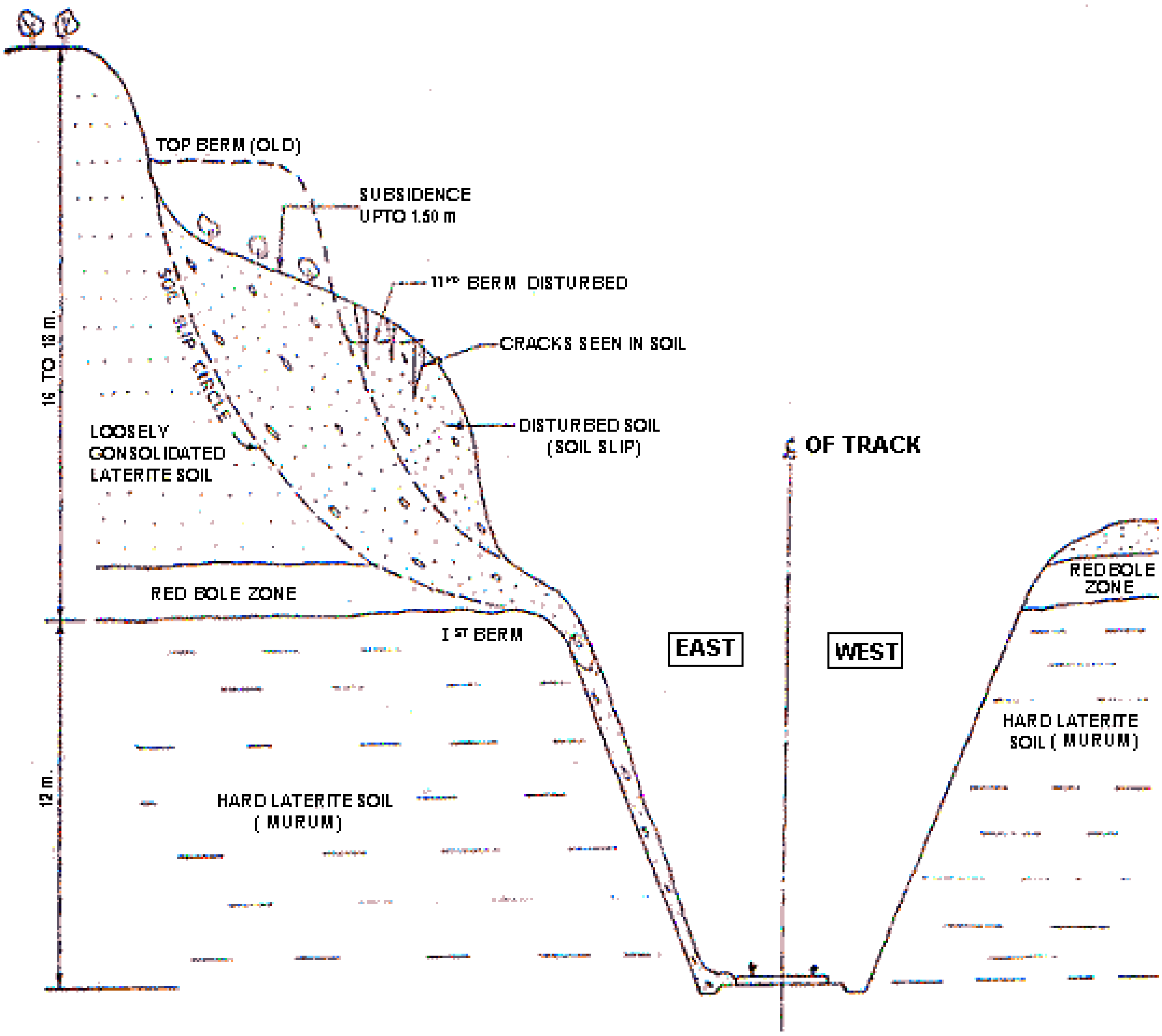


Fig. 6

CASE STUDY – 5

CROSS SECTION OF GOLAVALI CUTTING SHOWING SOIL SLIP FROM EASTERN HILL SLOPE AS ON 18.06.99



CROSS SECTION KM: 166 / 260

Fig. 7

CASE STUDY – 5

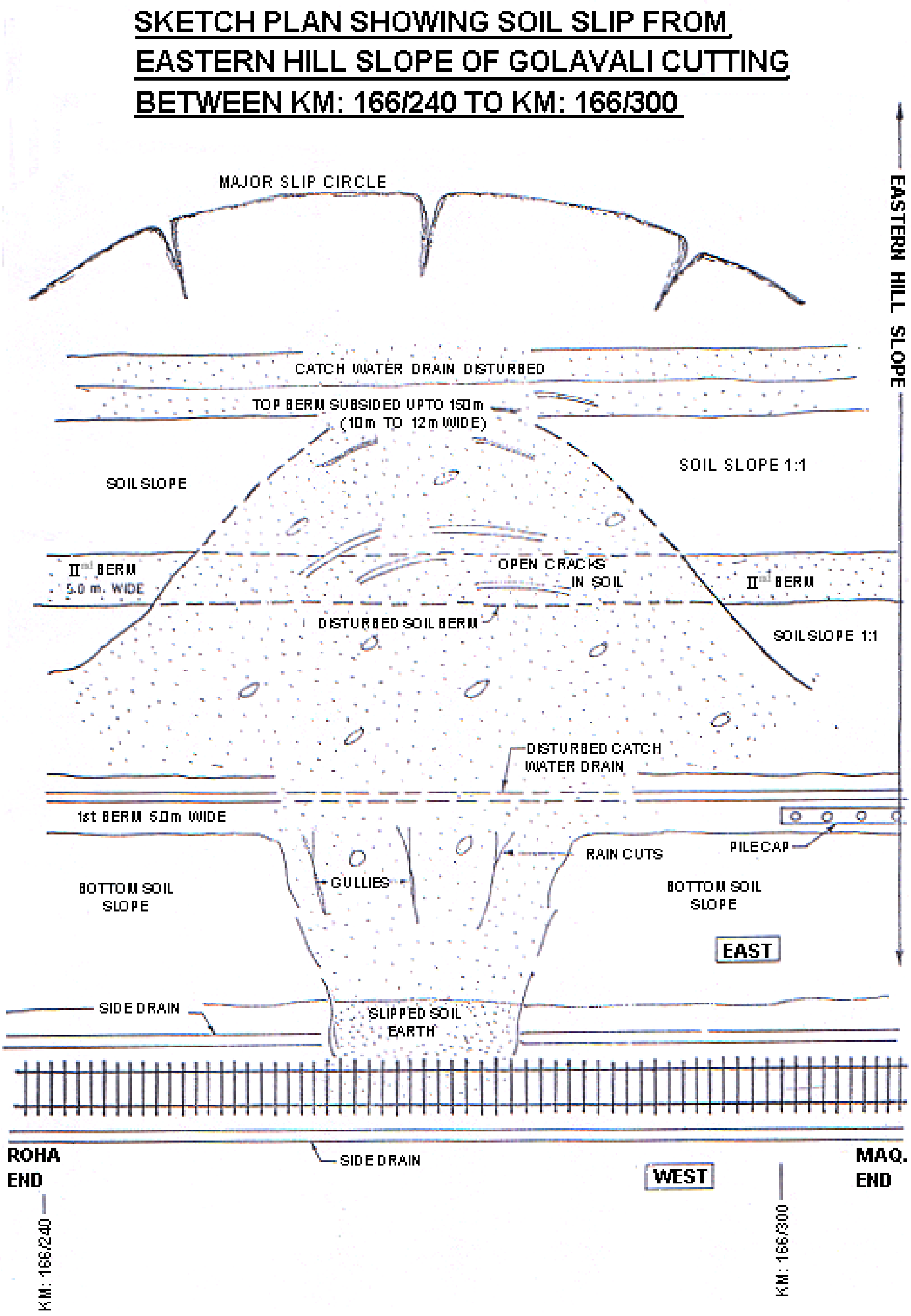
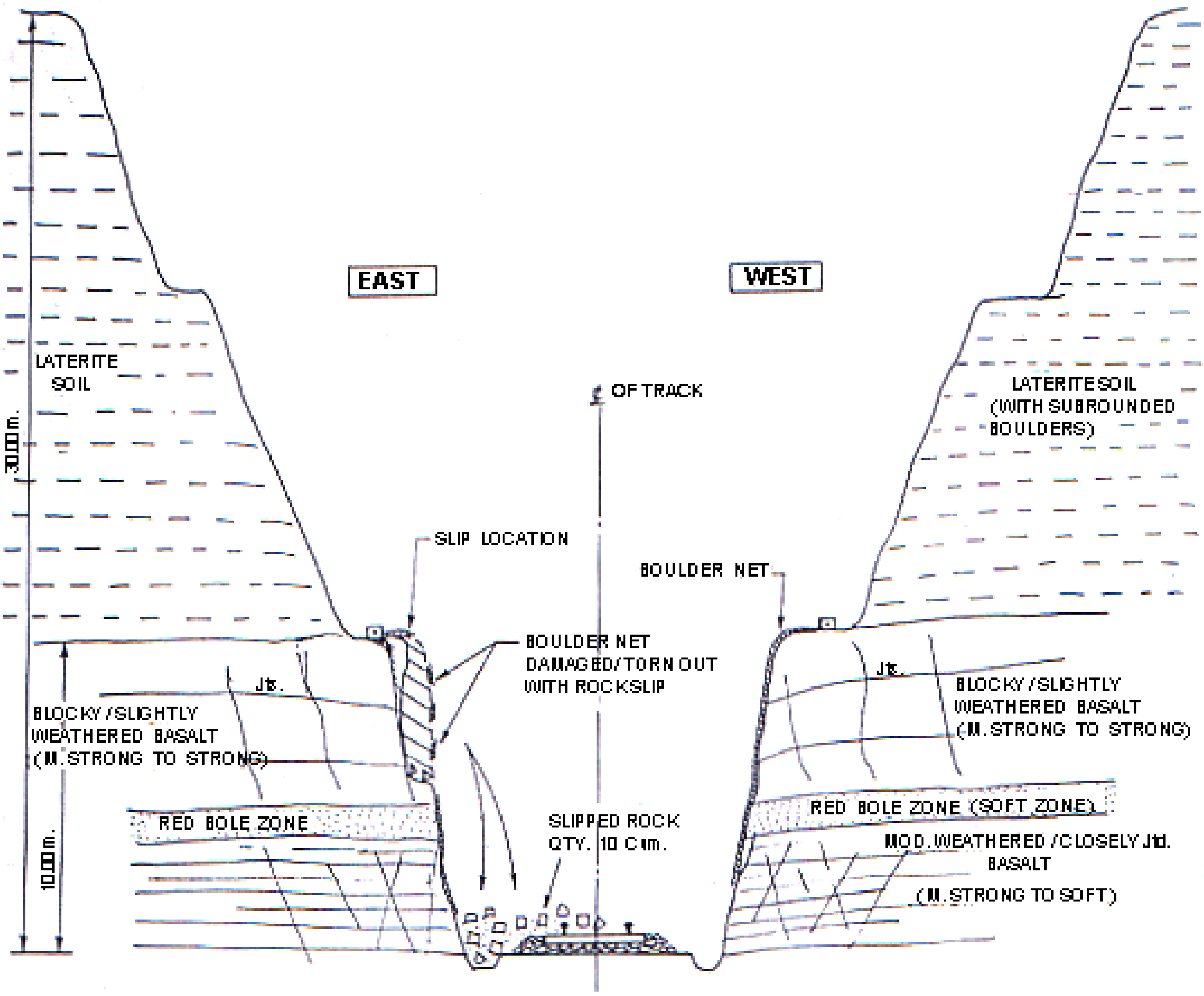


Fig. 8

CASE STUDY – 6

GEOLOGICAL SECTION OF KHEDSHI CUTTING AT ROCK SLIP LOCATION (KM: 199 / 615) (Dtd. 22. 03. 2000)



CROSS SECTION OF KHEDSI CUTTING AT 5 M U / S OF KHEDSI - II TUNNEL 'N' PORTAL KM : 199 / 615

REMARKS -:

1) ON Dtd. 22.03.2000 AROUND 2.25 Hrs. LOCO AND ONE COACH OF 6636 UP NETRAVATI EXP. TRAIN DERAILED AFTER HITTING THE BOULDERS WHICH SLIPPED FROM MIDDLE OF EASTERN HILL SLOPE, DAMAGING THE BOULDER NET INSTALLED OVER IT.

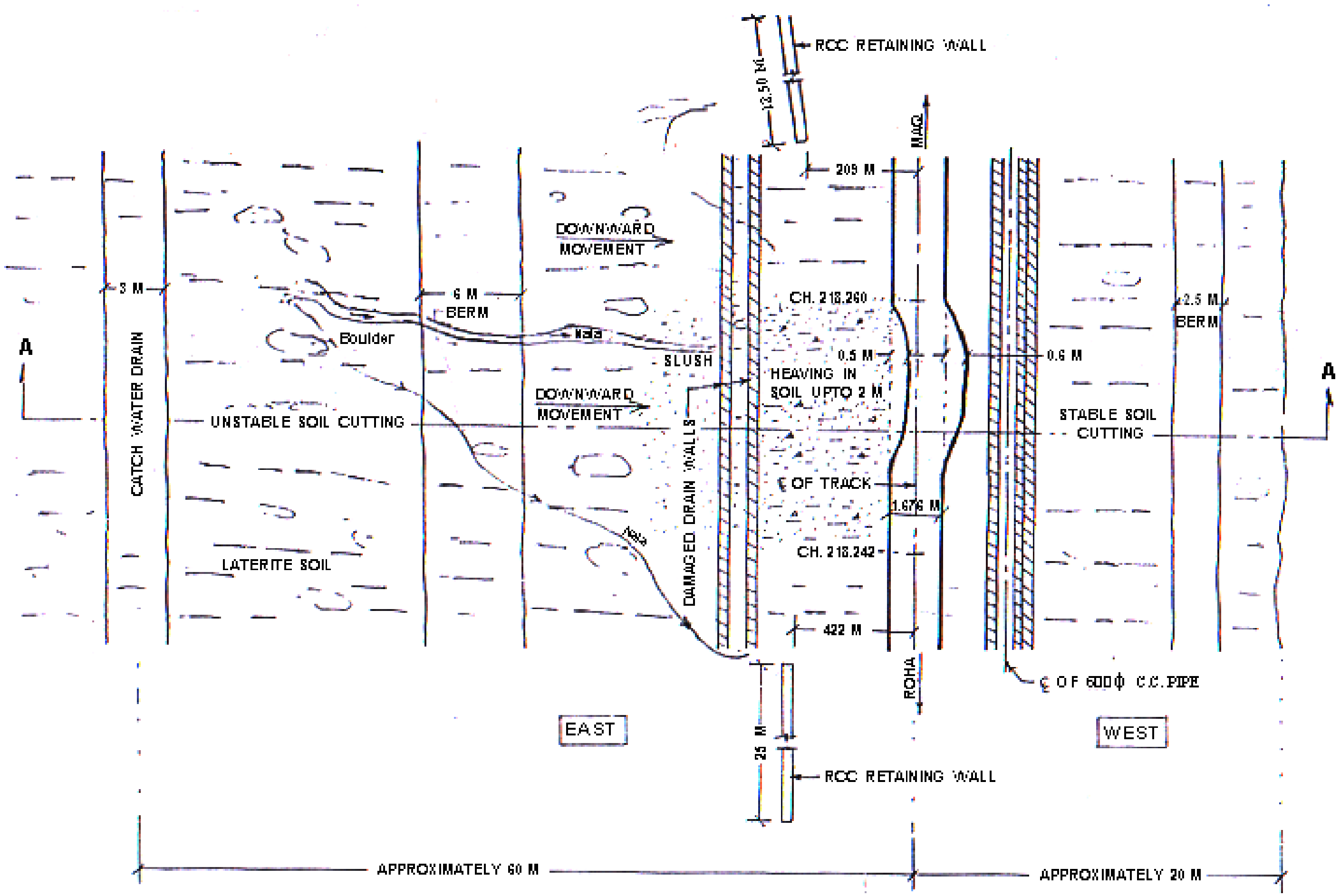
2) THE DERAILED TRAIN TRAVELLED UPTO A DISTANCE OF 339 m. AND STOPPED INSIDE KHEDSHI-I TUNNEL.

3) ONE BOULDER OF SIZE 1 m X 1 m X 0.8 m WAS DRAGGED BY DERAILED LOCO AND WAS SEEN TRAPPED BELOW IT.

Fig. 9

CASE STUDY – 7

KONDVI SOIL CUTTING



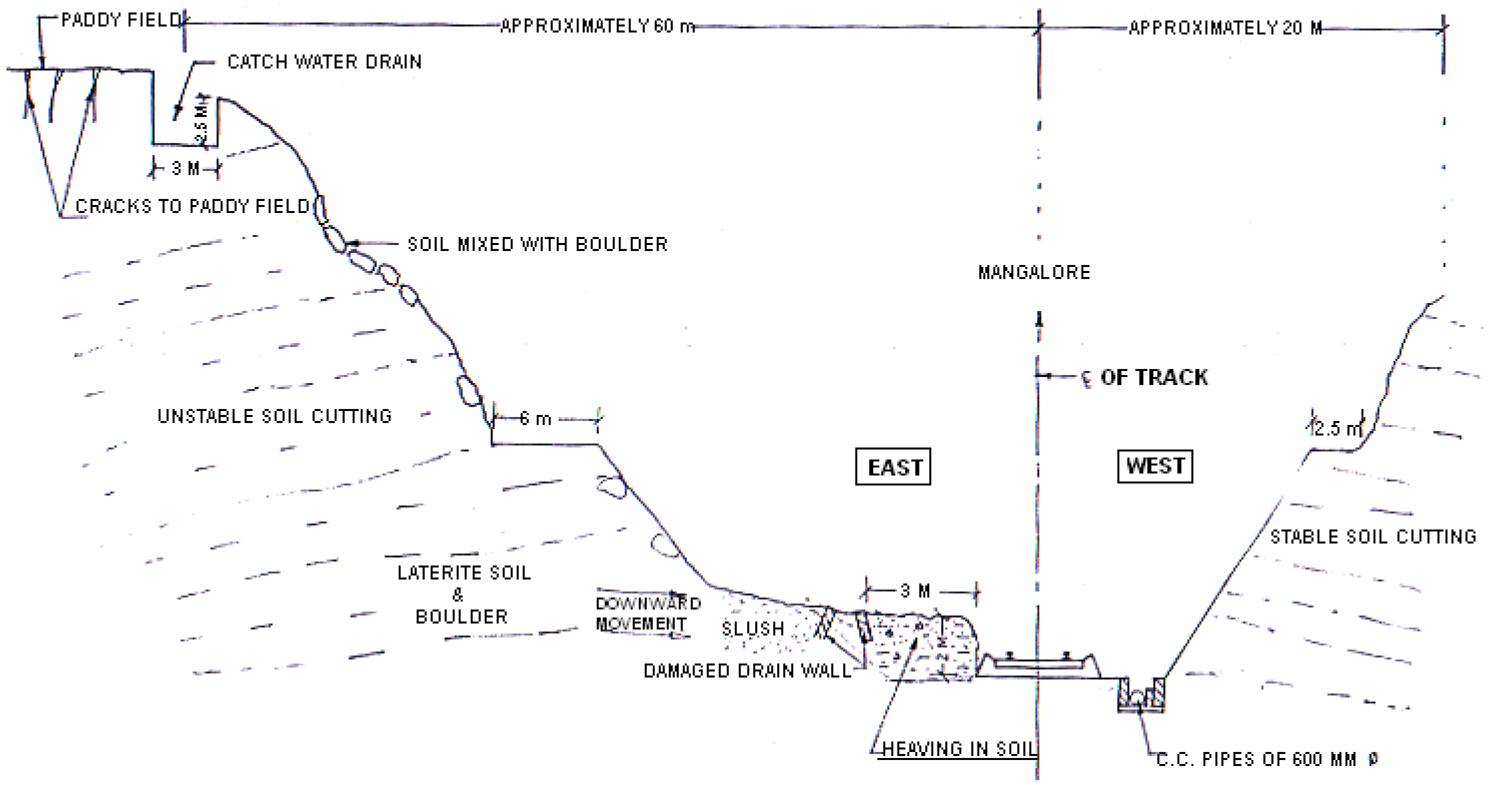
PLAN SHOWING HEAVING IN SOIL STRATA BETWEEN CH. 218.242 & CH. 218.260

AS ON JUNE END 97

Fig. 10

CASE STUDY – 7

KONDVI SOIL CUTTING



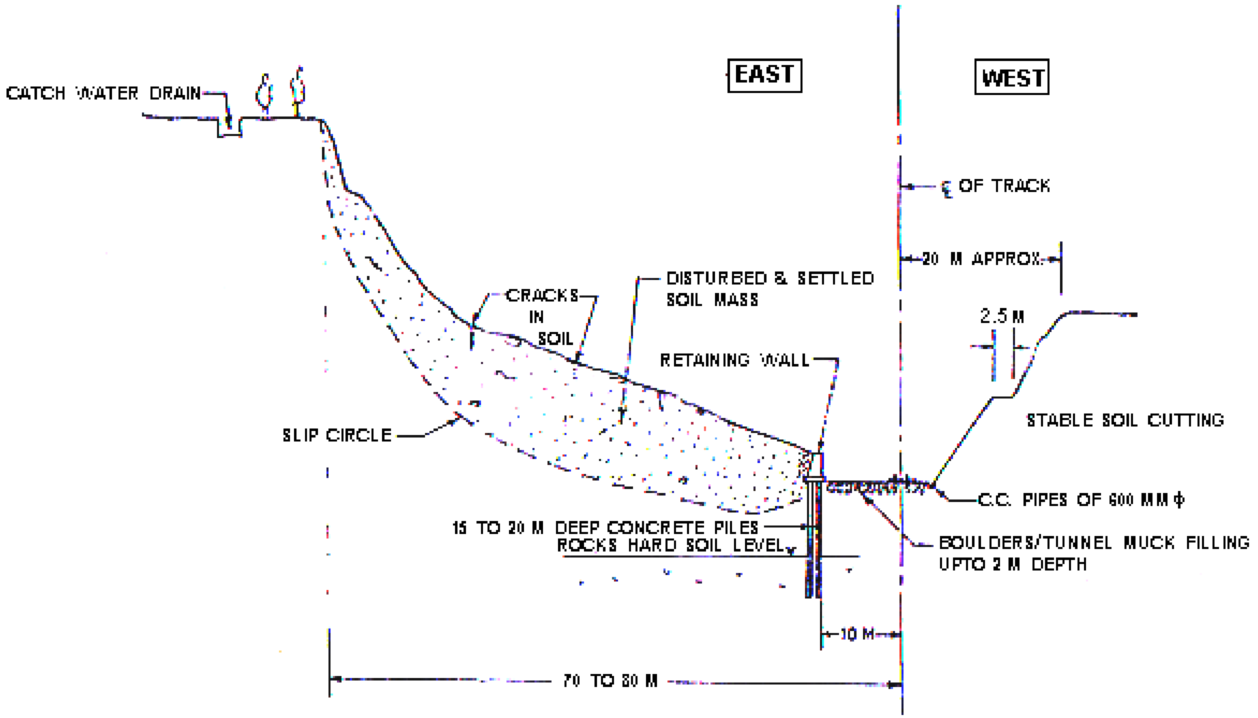
SECTION 'A-A'

CROSS SECTION AT CH 218.250 SHOWING HEAVING IN SOIL STRATA

Fig. 11

CASE STUDY – 7

KONDVI SOIL CUTTING

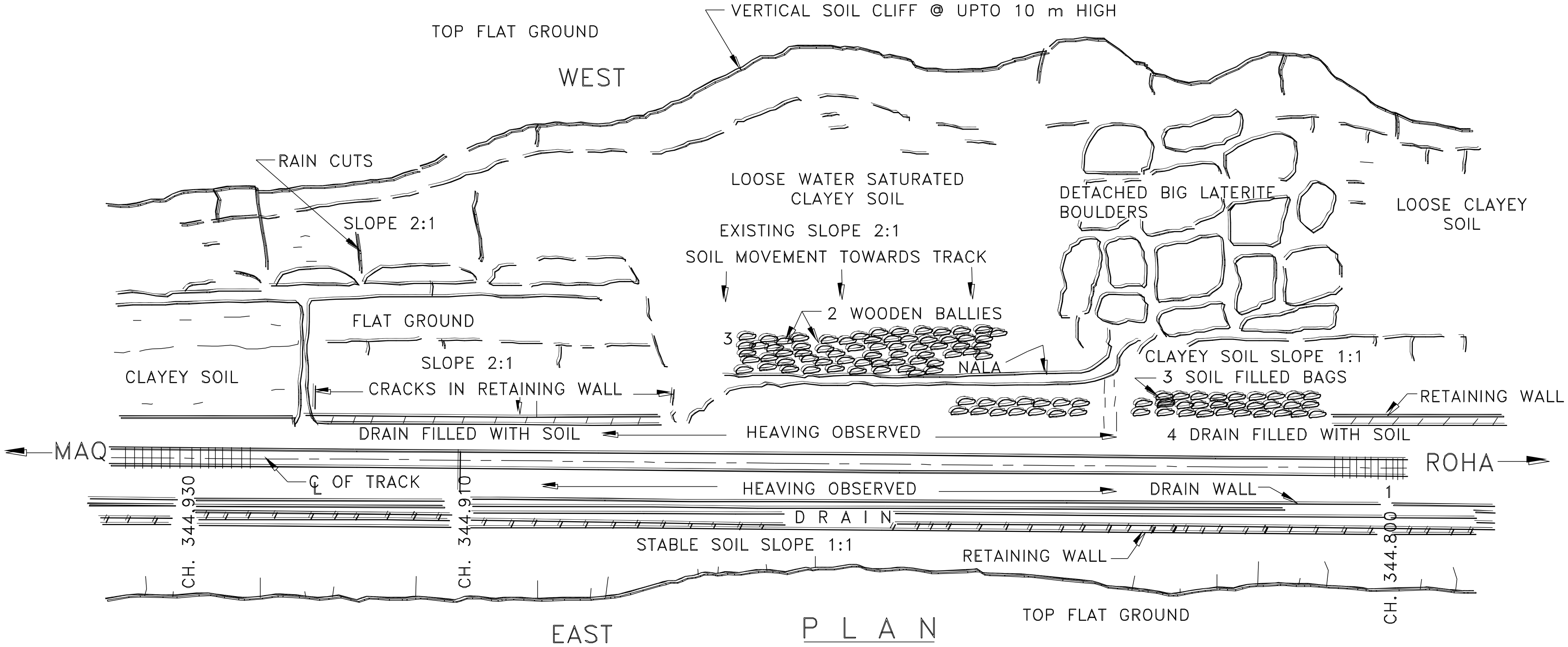


CROSS SECTION SUGGESTING REMEDIAL MEASURES TO STABILISE SOIL SLIP/HEAVING AT KONDVI

Fig. 12

CASE STUDY – 8

MIDC SOIL CUTTING, KUDAL



MEASURED SUGGESTED:

1. ARREST THE SURFACE WATER FLOW AND CHANNELISE THE WATER.
2. TAKE UP 30 m TO 40 m DEEP SHORING OF WOODEN BALLIES AT CLOSE SPACING.
3. PUT MUCK FILLED BAGS BETWEEN & BEHIND THE BALLIES ON BOTTOM SOIL SLOPE IN DIFFERENT ROWS.
4. REESTABLISH THE SIDE DRAIN ON WEST SIDE BY REMOVING SOIL/SILT FROM HEAVED UP AREA.

PLAN SHOWING SOIL COLLAPSE/MOVEMENT FROM WEST HILL SLOPE & HEAVING NEAR ADJOINING TRACK–AT CH. 344.800 TO CH. 344.930
MIDC SOIL CUTTING, KUDAL

Fig. 13

CASE STUDY – 9

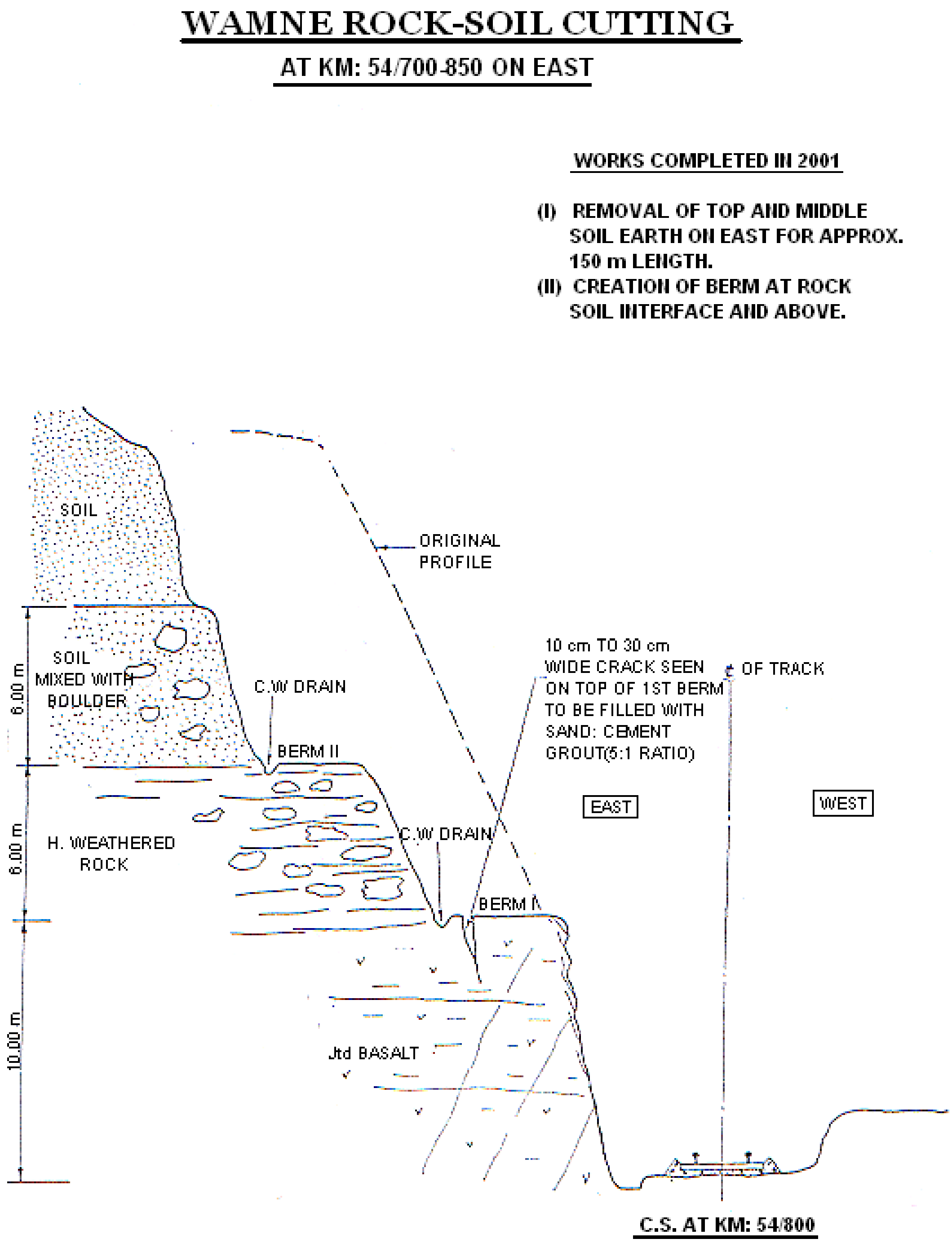


Fig. 14

Annexure–I

DESIGN OF CHANNELS / DRAINS

Model calculations for design of channels/drains are given below:-

DATA

Catchment Area (A) = 200000m² (20 hectares), assumed or taken from field
Rainfall Intensity (i) = 4 cm/hr
Runoff Coefficient (k) = 0.25 (assumed depending upon surface conditions)
Average particle size (d) = 0.005 m (D_{60})
General slope of the area (s) = 1 in 150 (ruling gradient)
Scouring velocity (v) = 3.0 m/sec. (take depending upon type of material)

DESIGN

Estimated Runoff (Q) = 1/36 k i A
k = Runoff Coefficient, i = Rainfall intensity, A= Area (in hectares)

$$Q = 1/36 \times 0.25 \times 4 \times 20$$
$$= 0.555 \ \text{m}^3/\text{sec}$$

Manning coefficient (η) = $1/24\ (d)^{1/6}$,

Where d = particle size (D_{60})

$$(\eta) = 1/24\ (0.005)^{1/6}, = 0.0172$$

Using Manning's Equation:-

$$Q = A.\ 1/\eta\ .\ R^{2/3}.\ S^{1/2}$$

Designing a efficient trapezoidal channel

$$R = y/2$$
$$A = y^2 \{2\ (1+m^2)^{1/2} - 1\},\ \text{Tan}60 = 1/m$$
$$m = 0.577$$
$$= 1.309\ y^2$$
$$Q = A.\ 1/\eta\ .\ R^{2/3}\ .\ S^{1/2}$$
$$0.555 = 1.309\ y^2 \times 1/0.0172 \times (y/2)^{2/3} \times (1/150)^{1/2}$$
$$0.555 = 6.78\ y^{8/3}, \quad y = 0.404$$

Base width (b) $= 2y\{(1+m^2)^{1/2} - m\}$,
$= 2 \times 0.404\{(1+0.577^2)^{1/2} - 0.577\}$
$= 0.476$, $= 0.50$m (take)

A $= 1.309\ y^2$, $= 0.200$

Velocity (V) $= Q/A$, $= 0.555/0.200$, $= 2.77$ m/sec < 3 m/sec

Hence safe.

Annexure – II

FACTORS TO BE CONSIDERED FOR PREPARATION OF FAILURE POTENTIAL EVALUATION MAP AND PROBABLE LANDSLIDE DISASTER MITIGATION MAP (FPEM & PLDM)

A. GEOLOGICAL FACTORS

i) Rock Condition

- Type/Lithology
- Nature of bedding its thickness and trend
- Nature of joints
- Nature of faults/thrust/warping
- Type of gauge material
- Type and extend of weathering
- Engineering properties of rocks formation

ii) Soil

- Type
- Thickness
- Engineering properties of soil

iii) Debris

- Thickness
- Soil and boulder percentage
- Shape of boulders

B. GEOMORPHOLOGICAL FACTORS

i) Slope

- Angle
- Nature
- Condition

ii) Nature of Erosion

- Uniform

- Channel type
- Toe erosion
- Piping action

C. HYDROLOGICAL FACTORS

- Rain fall intensity
- Drainage conditions
- Catchment area
- Run off characteristics
- Seepage conditions

D. MAN MADE FACTORS

- Blasting affect
- Deforestation
- Cultivation
- Nature of cut slope for track
- Any slope excavation work
- Other man made activities

Annexure -III

DO's & DON'Ts on the Use of Explosives

1. General

i) Do store explosives in dry, clean and well ventilated magazine.

ii) Don't smoke or have matches, naked light, etc. while storing, transporting, or using explosives.

iii) Don't keep explosives and detonators in the same box or the same magazine. Separate persons should carry explosives and detonators while transporting these to blasting site.

2. While Using

i) Do replace the cover of the case containing explosive after the required quantity has been taken out.

ii) Don't use tools made of iron or steel for opening cases. Use hard wood or brass implements.

iii) Don't leave explosives in the hot sun.

iv) Don't carry explosives in your pockets.

v) Don't make primers near large stacks of explosives.

vi) Don't insert any thing but a fuse in side a detonator.

vii) Don't handle or be near explosives during an electric storm. All person should retire to a place of safety.

viii) Don't use damaged or deteriorated explosives and accessories.

ix) Don't break explosive cartridge.

3. While Drilling and Charging

i) Do check conditions of shot hole with stemming rod before inserting detonating fuse.

ii) Do cut detonating fuse from the reel immediately after the primer has reached the bottom of the hole.

iii) Don't start drilling until you have made sure that the rock face contains no unfired explosives. Never drill into explosives

iv) Don't force detonating fuse into a hole

v) Don't keep large unwanted stocks of explosive near the shot holes

vi)	Don't	try to soften cartridge or hardened explosive by hitting or rolling on the ground.

4. While Stemming

i)	Don't	use metallic rods for stemming. Use only wooden rods. The end of the rod should be kept square by sawing off pointed end periodically.
ii)	Don't	apply pressure directly on the primer cartridge. Always put a few cm of stemming after the primer is in position inside the hole.
iii)	Don't	use sharp particles in the stemming.
iv)	Don't	damage fuse, lead wires or detonating fuse while stemming.

5. While Firing with Safety Fuse

i)	Do	use only approved crimpers for securing detonators on to fuse.
ii)	Do	use fuse lighters. If matches have to be used slit the end of fuse, hold the match head in the slit and rub the side of an empty match box against the match – head.
iii)	Don't	use short fuse. The minimum length should be 1.2m and make sure you have time to reach a place of safety before the explosive detonates.
iv)	Don't	use explosive cartridge for lightening fuse. It is extremely dangerous

.

6. While firing Electrically

i)	Do	keep the firing circuit insulated from the ground, bare wires, rails, pipes or any other paths of stray currents.
ii)	Do	test the firing circuit with an Ohmmeter or circuit tester from the firing points.
iii)	Do	make sure that all joints are firm, clean and dry.
iv)	Do	keep lead wires short-circuited until ready for firing.

v) Don’t use electric detonators during dust storms or near any other source of large static charges.

7. Before and After Firing

i) Do handle misfire with care.

ii) Don’t fire until you have made sure that all surplus explosives have been removed and all persons, vehicles and equipment are at a safe distance.

iii) Don’t return to the blasting site soon after misfire. Wait five minutes when firing electrically or wait 30 minutes if firing with safety fuse.

Annexure- IV

PROFORMA FOR COLLECTION OF DATA REGARDING LANDSLIDE/ROCKFALL OCCURRENCE AND CLEARENCE

1. Location of slide:

i) Name of section/route

ii) Name of the place

iii) At Km From To

iv) Name of slide, if any

2. Data to be collected regarding the slide when it is active:

i) Date of sliding

ii) No. of times sliding has taken place in the year

iii) Duration for which traffic was blocked by the slide no. of days/hours

iv) Damage to property/persons

v) Quantity of material cleared

vi) Method of clearance

vii) Cost of clearance operation

viii) Were any permanent stabilizing measures executed since last sliding and if so their efficacy

ix) Is the slide preceded by rainfall

x) Is extent of area participating in sliding

I. Confined to up hill slide of rail road only

II. Confined to downhill slide of railroad only

III. Covers both

xi) Is the cause of slide due to man made causes such as back cutting etc.

xii) Does the slide appear to be a superficial one or deep seated one

3. Standard information/ Data to be collected about the slide

3.1 *Prepare a sketch of the slide area covering the slope both uphill and downhill of the rail road and include the following informations*

i) Length of slide from crown to toe

ii) Width of slide

iii) Maximum depth

iv) General description

- Slide area
- Condition of slopes
- Presence of erosion gullies
- Presence of water springs
- Tension cracks

3.2 Geological Data

i) Nature of rock

ii) Type of rock and formation

iii) Dip and Strike

iv) Weathering

3.3 Geotechnical Data

i) Nature of soil

ii) Alternations of force acting

iii) Action of water

3.4 Causes of slide

i) Geological causes

ii) Geotechnical causes

3.5 Classification of slide

3.6 Remedial measures

3.7 Recommendations

Annexure-V

Proforma of the Register of cuttings

Particulars of Cuttings

1. Name of section: -
2. Name & No. of cutting: -
3. Kilometer From To
4. Max depth of cutting: -
5. Initial side slope of cutting: -
6. Degree of curvature if any: -
7. Type of strata: -
8. Past History of cutting: -
9. General Remarks: -

Inspection Details

PWI Inspection		AEN Inspection		Remarks of DEN / Sr. DEN / THOD	Action Taken by PWI/IOW
Inspection details	Signature with date	Inspection details	Signature with date		

Annexure-VI

Check list of points to be examined during inspection of cutting

1. Safety aspects
    - Safety precautions at slides/slips, breaches,
    - Damaged culverts/bridges
    - Branches of trees with low height obstructing vertical clearance
    - Precariously perched trees on top of cutting and slopes
    - Adequate and condition of trolley refuges within cutting
2. Land Slide/unstable areas
    - Behaviour of the slide whether dormant, active or unstable areas likely to become active, any movements observed
    - Effectiveness of control measures already undertaken
    - Functioning of drainage arrangement
    - Condition of protective/control structures and their effectiveness
3. Snow-fall/damages
    - Advance action required for snow clearance
    - Condition of snow/avalanche control structures-repairs required
    - Additional control measures required
    - Snow fall pattern
4. Cut section
    - General condition of existing protection/stabilization measures
    - Requirement of protection/stabilization measures to retain/correct slope
    - Easing of slopes required or not
    - Width of berm adequate or not
    - Whether berm provided at soil/rock interface
    - Presence of tension cracks
    - Erosion on slopes
    - Loose boulders and overhangs
    - Blockage of subsurface drainage system e.g. horizontal drains
5. Drainage (Side, Catch water drains, etc.)
    - Whether provided or not
    - Are the existing drainage facilities maintained in good order
    - Cross sectional area adequate or not
    - Blockage/depressions/damage to drains
    - Flow in drain and disposal of discharge
    - Longitudinal slopes,
    - Lining etc., - in good order or to be rectified
6. Culverts/minor bridges
    - Damages, if any

- Free flowing or is there any blockage
- Silting of catch pits
- Damages to head walls, parapets, etc., if any

7. Retaining walls, breast walls etc

- General condition
- Any damages/cracks
- Weep holes – effective functioning
- Pitching - properly maintained or not

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