"The key to growth is the introduction of higher dimensions of consciousness into our awareness."

– Lao Tzu

LABORATORY STUDIES 5

For a sound engineering geological investigation proper understanding of material characteristics is pre-requisite. Further, study of natural hazards like landslide accounts for number of aspects and their variables that are normally accomplished through intensive field study of various terrain parameters with duly supported maps, collateral satellite data, data from software based analysis, etc. However, precise inference on material aspect can only be derived, when material properties and their behavioural response under stress conditions are determined.

In the present case also intense inputs from laboratory studies have been obtained, which are considered to be of utmost importance for carrying out Stability Analysis of Landslides, Micro-Level Landslide Hazard Zonation Study and for suggesting Mitigatory Measures. This present chapter deals with important engineering properties of the large number of disturbed and un-disturbed soil and rock samples, collected during the various stages of field investigations.

SAMPLING

As it has already been eluded that the Mangti landslide is characterized by a deep multi-rotational type of slip movement, predominantly confined to colluvial materials. Therefore, in order to study the nature and composition of slumped mass, its lateral continuity and vertical variations; a detailed sampling plan has been developed (Figure 5.1). The sampling plan has been envisaged to obtain representative bulk samples from various parts of the landslide viz. Crown, Crown Scarp, Zone of Ablation, Landslide Body Mass, Side Scarps, Toe portion as well as from un-disturbed shoulder regions. For this Indian Standards specified core cutter (Volume 1021 cc) has been used.

As the postulated slip plane is confined beneath the debris and within the in-situ country rock (Biotite Gneiss), representative block samples of Biotite Gneiss have also been collected for determining its engineering properties.

Absence of drilling activity was the biggest constraint in working out the depth wise (vertical) variation in slumped mass and the base of the landslide. Therefore, sampling for this has been done to the extent possible (≤ 3m), through manually excavated pits, particularly dug for Piezometer installation.

The samples were tested in various geotechnical laboratories viz. National Hydel Power Corporation (NHPC) Dhauliganga Project at Chirkila, Pithoragarh, Uttarakhand; Gujarat Engineering Research Institute (GERI) at Vadodara; Department of Earth Sciences, I.I.T. Mumbai; and Geo-test House at Vadodara.

ENGINEERING CHARACTERISTICS OF SOIL

Engineering properties of any soil regolithic mass depends on its basic mineralogical composition, particle size and moisture content. The variation in these basic characteristics in turn affects its all over strength parameters. Important physical and engineering properties of the regolithic mass determined are –

1. Granulometric Characteristics
2. Atterberg Limit's

3. Moisture Content and Density
4. Permeability
5. Cohesion, Angle of Internal Friction and Shear Strength

Details of the tests carried out on both disturbed and un-disturbed soil samples is elaborated independently as under –

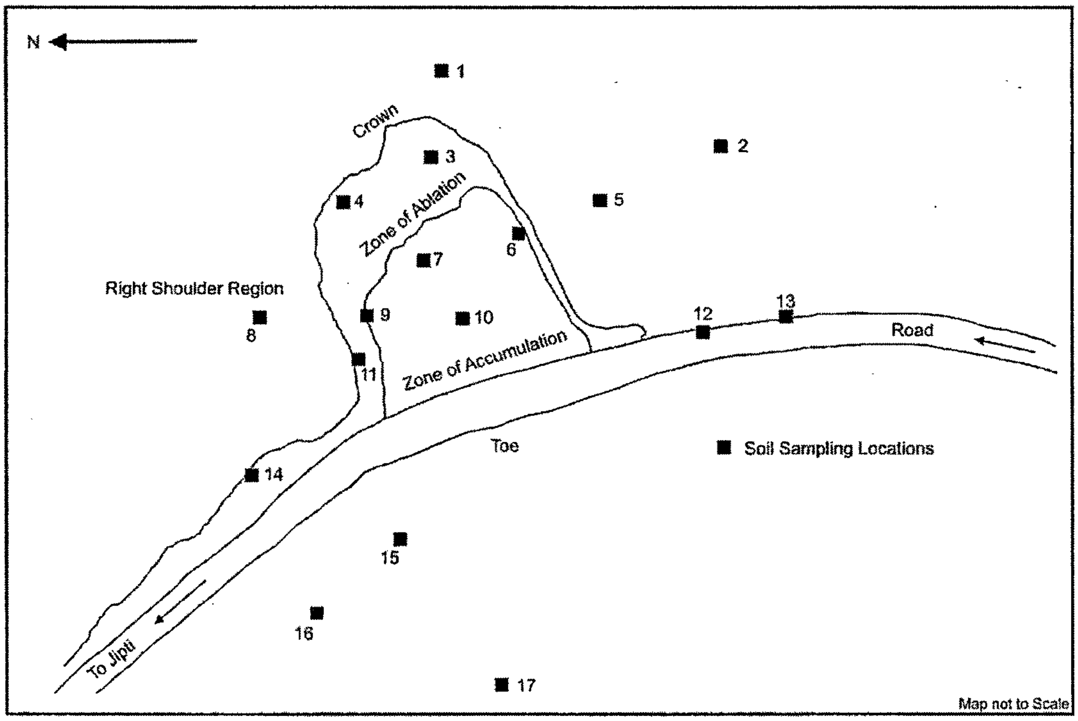

Figure 5.1: Schematic Plan Depicting Sampling Locations of Disturbed Soil Samples, Mangti Landslide

GRANULOMETRIC CHARACTERISTICS

Granulometric or Gradation characters of sediments denotes the distribution pattern of size of grains in a collected sample. Gradation analysis of the various samples has been done using standard procedure specified in Earth Manual (USBR, 1965). Data obtained through sieving were then plotted and interpreted using GEOSYSTEM FIELD AND LABORATORY TESTING Software Program, especially designed to construct standard granulometric plots and derive interpretations for the various geotechnical properties. Based on plotted grain size curves using particle size and percent of fines, the soil groups (USCS, 1952) were identified and sample specific co-efficients viz.

(1) Co-efficient of Uniformity, $C_U = D_{60} / D_{10}$

(2) Co-efficient of Curvature, $C_C = (D_{30})^2 / D_{10} \times D_{60}$

were computed to obtain relevant soil groups.

To be well graded (GP or SW), the C_C must be between 01 and 03 and in addition, the C_U must be greater than 04 for gravels and greater than 06 for sands (USBR, 1965). Grain size distribution plots along with other relevant information for the various analyzed samples are given as **ANNEXURE – 4**. The summary on obtained granulometric parameters is given in Table 5.1. It could be seen from the tabulated data that majority of collected disturbed surface samples show mixed characters and fall within the groups of **SP-SM** of Unified Soil Classification System (USBR, 1965), signified by " *poorly graded sand, gravelly sands with little or no fines (SP) and silty sands, poorly graded sand – silt mixtures (SM).*" Further the determined Co-efficient $\mathbf{C_U}$ and $\mathbf{C_C}$ ranges between **60.56 – 11.84** and **0.99 – 0.29** respectively.

Granulometric study of undisturbed samples collected from the various Piezometer pits (Figure 4.11) show considerable homogeneity in sediment/soil nature, vertically as well as laterally and by and large marked with single type of soil horizon (Table 5.1). The sediments are predominantly characterized by "poorly *graded sands, gravelly sand mixtures, little or no fines (SP)*"; the plotted Granulometric curves for the various Piezometer pit samples are given as **ANNEXURE – 4.**

Table 5.1 - Gradation & Size Characteristics of Colluvial Soil Sediments, (Disturbed/ Un-disturbed) Mangti Landslide

Sample No	Depth in (cm)	Material Nature	C_u	C_c	USCS * Group
1	75	Colluvial	41.38	0.37	**SP - SM**
2	100	Colluvial	45.97	0.32	**SP - SM**
3	100	Colluvial	60.56	0.29	**SP - SM**
4	100	Colluvial	17.31	0.70	**SP - SM**
5	85	Colluvial	-	-	**SM**
6	100	Colluvial	29.24	0.46	**SP - SM**
7	100	Colluvial	19.48	0.51	**SP - SM**
8	80	Colluvial	28.23	0.38	**SP - SM**
9	150	Colluvial	11.84	0.37	**SP**
10	100	Colluvial	31.60	0.43	**SP - SM**
11	90	Colluvial	38.25	0.32	**SP - SM**
12	90	Colluvial	22.44	0.50	**SP - SM**
13	97	Colluvial	18.84	0.50	**SP - SM**
14	100	Colluvial	17.47	0.47	**SP - SM**
15	80	Colluvial	35.75	0.36	**SP - SM**
16	100	Colluvial	32.98	0.32	**SP - SM**
17	100	Colluvial	42.30	0.99	**SP - SM**
Piezometer Pits Samples (Un-disturbed)					
PZ- 1	170	Colluvial	12.63	0.32	**SP**
PZ- 2	180	Colluvial	16.14	0.26	**SP**
PZ- 3	130	Colluvial	20.72	0.25	**SP**
PZ- 4	140	Colluvial	20.36	0.30	**SP-SM**
PZ- 5A	120	Colluvial	19.75	0.30	**SP**
PZ- 5B	210	Colluvial	19.80	0.28	**SP**
PZ- 6	160	Colluvial	22.76	0.30	**SP**
PZ- 7	Location is situated on river terrace comprising rounded clasts > gravel (GP) with coarse sand matrix				

* USCS-Unified Soil Classification System (USBR, 1965)

ATTERBERG LIMIT'S

Physical properties of fine grained soils are greatly affected by the water content. Particularly clayey soils are soft in viscous state and hard in dry state. Between these two extremities soil can be moulded and formed without any cracking or rupturing the soil mass, i. e. plastic condition. These moisture dependent behavior of soil mass has been designated as the limits of consistency, which are called "Atterberg limits" after a Swedish soil scientist, are used in the Unified Soil Classification System (USBR, 1965) as the basis for laboratory differentiation between materials having grain size > 200 mesh and characterized by appreciable plasticity (clays) and slightly plastic or nonplastic materials (silts). The soil consistency limits are evaluated by determining the Liquid Limit (LL), Plastic Limit (PL) and the difference between two termed as Plasticity Index (PI). The plasticity index shows the range of water contents within which the soil is plastic. Highly plastic soils have high PI values, whereas in a nonplastic soil, the PL and LL are the same and the PI equals 0 (USBR, 1965). 'LL', 'PL' and 'PI' values of undisturbed soil samples collected from Piezometer Pits were determined as per procedures cited in Indian Standard IS : 2720 (Part 5) – 1985. Results obtained for various Piezometer pit samples shows that all of them are non-plastic sediments having very less or no fines (Table 5.2).

Table 5.2 - Soil Consistency and Permeability Parameters of Piezometer Pit's Samples, Mangti Landslide

Sample ID	Atterberg's Limit			Permeability 'k' in cm/sec
	Liquid Limit %	Plastic Limit %	Plasticity Index %	
Pz-1	37.47	Non-Plastic	Non-Plastic	0.41×10^{-5}
Pz-2	26.70	Non-Plastic	Non-Plastic	0.18×10^{-5}
Pz-3	29.65	Non-Plastic	Non-Plastic	8.60×10^{-5}
Pz-4	35.70	Non-Plastic	Non-Plastic	8.41×10^{-5}
Pz-5/A	36.50	Non-Plastic	Non-Plastic	4.93×10^{-5}
Pz-5/B	33.73	Non-Plastic	Non-Plastic	0.24×10^{-5}
Pz-6	34.00	Non-Plastic	Non-Plastic	7.31×10^{-5}

MOISTURE CONTENT AND DENSITY

Moisture content is expressed as a percentage of the dry weight of the soil and is most influential factor affecting the properties of a soil. It is also the principal factor, which is subject to change either from natural causes or anthropogenic interventions.

Similarly Density or Unit Weight of a unit volume of soil is considered as the basic parameter to which all other material performance characteristics are related. Density is determined in different expressions viz. natural density, remolded natural density, wet density, dry density and Proctor maximum dry density. However, natural dry density is having its wide application in stability computation for determining the overburden pressure. Moisture content (w) and Dry density (γ_d) parameters of collected disturbed samples were determined as detailed in Earth Manual (USBR, 1965). The obtained parameters are given in Table 5.3.

Dry density (γ_d) ranges between **1.0217 – 1.7263 gm/cm^3** whereas, the **Bulk density (γ_m)** varies from **1.8036 gm/cm^3 to 1.0696 gm/cm^3**.

Moisture Content (w) also shows large variation ranging from **2.08% to 9.60%**. This obtained range of variation is attributed to sample specific change in relative proportion of fines and its depth.

In-situ samples collected with the help of a core cutter from Piezometer pits were analyzed for determining **Proctor Maximum Dry Density (γ_{max})** as per IS: 2720 (Part – VII) – 1980. The results obtained show Maximum Dry Density (Table 5.4) ranging between **1.861% gm/cm^3 and 2.07 gm/cm^3** with **Optimum Moisture Content** ranging between **7.00% and 12.00%**.

Table 5.3 - Bulk Density, Dry Density & Moisture Content of Disturbed Soil Samples, Mangti Landslide

Sampling Core Dimensions:

Inner Diameter of the core: 10.2 cms. Length of the core: 12.5 cms.

Volume of the Core Sample: $\pi r^2 l$ =3.14 X $(5.1)^2$ X 12.5 =1021.82 cm^3

Sample ID	Sampling location	Weight Bulk Sample (gms)	Weight of Bulk Sample	Bulk Density (gm/cm^3)	Dry density (gm/cm^3)	Moisture Content (%)
1	Top of The Present Slide Zone	1680	1617	1.6441	1.5824	3.89
2	Right Most of the Slided Zone	1440	1382	1.4092	1.3524	4.19
3	Escarpment Right Top	1430	1399	1.3994	1.3691	2.21
4	Escarpment Right Top	1400	1407	1.4092	1.3769	2.34
5	Right Peripheral	1093	1044	1.0696	1.0217	4.69
6	Escarpment Right Shoulder	1777	1662	1.7390	1.6265	6.91
7	Escarpment Middle	1577	1503	1.5433	1.4709	4.92
8	Left Peripheral	1596	1512	1.5619	1.4797	5.55
9	Escarpement Right	1843	1764	1.8036	1.7263	4.47
10	Regolithic Mass Centre	1318	1228	1.2898	1.2017	7.32
11	Escarpment Left shoulder	1575	1509	1.5413	1.4767	4.37
12	Right Side of L.S. at some Distance	1734	1582	1.6969	1.5482	9.60
13	Right Side of L.S At Significant Distance	1588	1512	1.5540	1.4797	5.02
14	Left Side of L.S At Some Distance	1550	1455	1.5169	1.4239	6.52
15	Toe of the Slided Zone Middle Scarp	1453	1396	1.4219	1.3661	4.08
16	Toe of the Slided Zone Left Side	1700	1652	1.6636	1.6167	2.90
17	Toe of the Slided Zone the Stream	1716	1681	1.6793	1.6451	2.08

Table 5.4 - Proctor Density Test Results of Piezometer Pit Samples (Un-disturbed), Mangti Landslide

Sample ID	Standard Proctor Density Test	
	Maximum Dry Density (M.D.D)	Optimum Moisture Content (O.M.C)
Pz-1	1.928	12.00
Pz-2	2.070	07.00
Pz-3	1.978	10.00
Pz-4	1.959	09.96
Pz-5/A	1.959	09.65
Pz-5/B	1.968	09.00
Pz-6	1.861	12.00
15	2.020	09.30
13	2.006	09.80

PERMEABILITY

Interstitial spaces in a soil mass apart from providing mechanism for compressibility facilitate the groundwater to move through the soil mass. Further the state of water movement is called percolation; the measure of it is called permeability; and the factor relating permeability to unit conditions of control is called the co-efficient of permeability (USBR, 1965).

The co-efficient of permeability (k) is based on Darcy's Law that states *"The flow rate of fluid through homogeneous porous media is directly proportional to head loss and inversely proportional to the length of flow path."*

Laboratory permeability has been determined for various Piezometer pits' samples in accordance with procedure laid under Indian Standard IS: 2720 (Part 17) – 1986, adopting Falling Head Test method and using the following relation (Equation 8) –

$$k = 2.303 \frac{aL}{A\,(t_f - t_i)} \log_{10} \frac{h_1}{h_2} \qquad (8)$$

Where, a = Area of the stand-pipe (cm^2),

L = Length of specimen (cm),

A = Area of specimen (cm^2),

t_f = Final time 't'; t_i = Initial time 't' in sec,

h_1 = Initial Head 'h' and h_2 = Final Head 'h' in cm.

Soil permeability tends to change from material to material as well as laterally and vertical. The range of permeability (USBR, 1965) varies between 10^{-4} cm/sec, Very High (clean Gravel) to 10^{-5} cm/sec, Very Low (Massive Clay).

The obtained permeability results (Table 5.2) show variation ranging from 8.60×10^{-5} to 0.18×10^{-5} cm/sec that are in conformation with the Granulometric characteristics of soil, i.e., predominantly of SP – SM Groups.

COHESION, ANGLE OF INTERNAL FRICTION AND SHEAR STRENGTH

The engineering computations concerned with the strength of a soil deals primarily with its shearing strength, i.e., the resistance to sliding of one mass of soil against another. Shearing Strength (S) of soil mass has two major components (Equation 3);

(i) Amount of stress (σ) normal to the shearing plane and

(ii) Internal friction (tan $\emptyset$) and Cohesion (c).

$$S = C + \sigma - U \tan \emptyset \qquad (9)$$

Where, U = Pore water Pressure.

Measurement of shearing strength in the laboratory is normally accomplished either by the Direct Shear Test or by the Tri-axial Test.

Direct Shear Test has been conducted on various collected soil samples in accordance with the Indian Standard IS: 2720 (Part 13) – 1986. Observed shear stress data were plotted against applied normal stress to obtain Shearing Strength, Cohesion and Angle of Internal Friction. The obtained plots are given as Figure 5.2.

Results computed through Box Shear Test (Table 5.5) shows cohesion **'c'** ranging from **10 – 18 KN/m²**; angle of internal friction **'Ø'** between **26° 42' – 32°**; and Shear Strength **'S'** between **91 – 130 KN/m²**. These parameters have been subsequently used for carrying out stability analysis of the studied landslides.

Table 5.5 - Box Shear Test Parameters of Piezometer Pit's Samples, Mangti Landslide

Sample ID	Box Shear Test		
	Cohesion 'c' in KN/m²	Angle of Internal Friction 'Ø' in Degrees	Shear Strength in MPa
Pz-1	12	29.0	0.120
Pz-2	13	29.0	0.120
Pz-3	15	26.7	0.091
Pz-4	17	28.0	0.096
Pz-5/A	18	31.0	0.125
Pz-5/B	12	27.0	0.119
Pz-6	13	30.0	0.120
15	10	32.0	0.110
13	15	28.0	0.130

ENGINEERING CHARACTERISTICS OF ROCK

The Mangti landslide's talus/colluvial material comprises of a considerable amount of massive sized boulders of biotite gneiss rock that have got detached from the country rock as a result of incipient weathering over a long period of time. The rocks when subjected to displacement under loading conditions may be damaged, i.e., they may produce cracks and break into smaller fragments. The possible effects of load on rocks depend greatly on the physical and mechanical properties of rocks. Thus, it becomes quite necessary to assess these properties, so as to facilitate in deriving a qualitative and quantitative assessment of the condition of in-situ rocks occurring at the rupture surface of the active landslide zone.

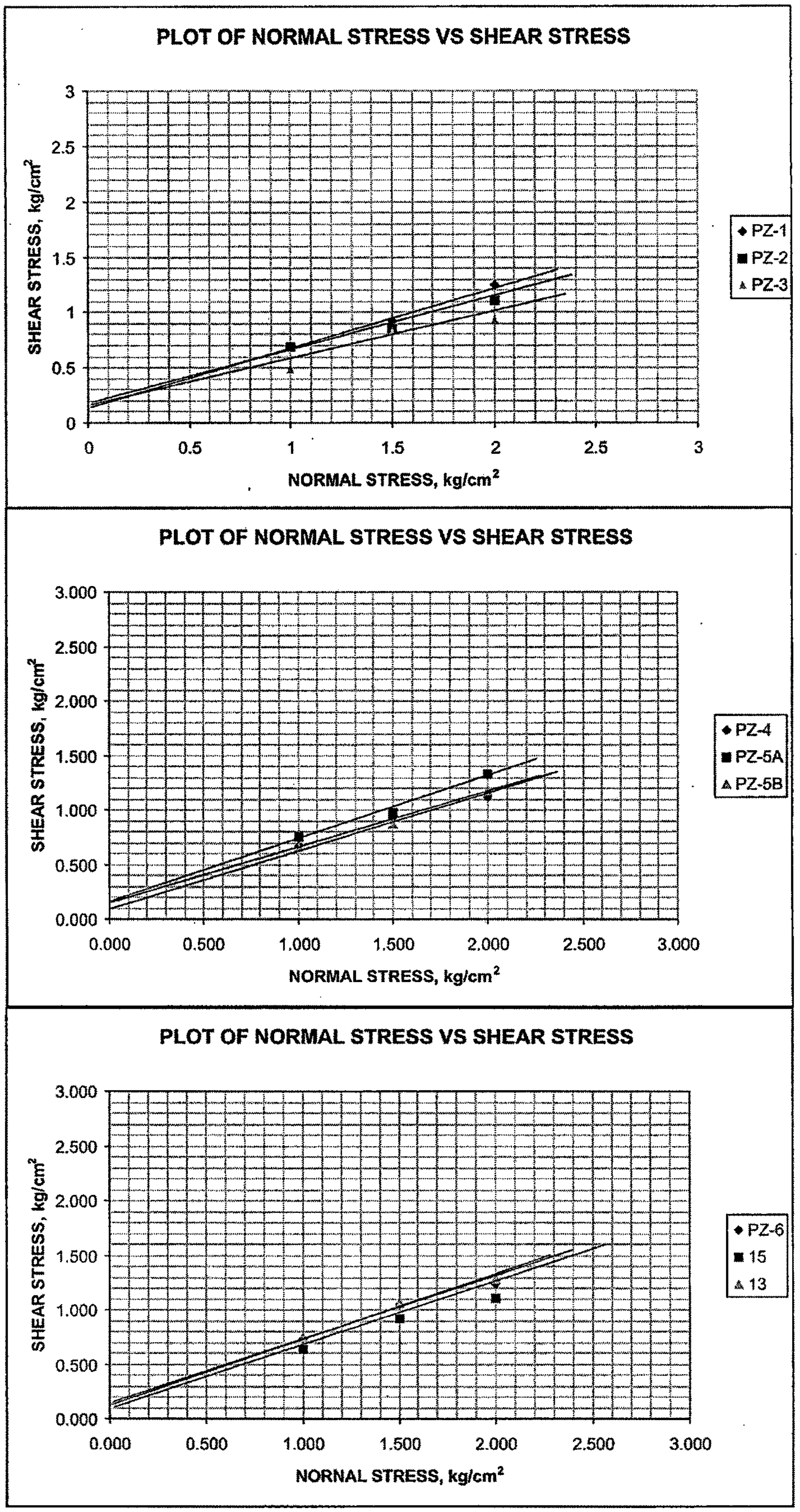

Figure 5.2 - Shear Test Plots of Piezometer Pit's Samples, Mangti Landslide

PHYSICAL PROPERTIES OF BIOTITE GNEISS

In order to determine various physical and geo-mechanical properties of biotite gneiss rock, first block samples were subjected to coring and number of cylindrical core samples were extracted using laboratory drill machine. The adopted dimensional parameters of the core samples, i.e., radius (27mm), length (108mm) were such that the specimen can be used in for testing tri-axial compressive strength also. All relevant physical properties viz. Bulk Density, Dry Density, Absorption of Water Content, Bulk Specific Gravity, and Porosity were determined in accordance with the Indian Standard Codes of Practices.

Determination of Dry Density, Bulk Density and Porosity

A total number of 06 samples have been tested as per Saturation and Caliper Techniques elaborated in IS 13030 : 2001. The results of this test are tabulated in Table 5.6. Following observations can be deduced from the test results –

1. **Dry Density** values of the Biotite Gneiss rock at Mangti Landslide varies between 2.66 and 2.75 gm/cc (Table 5.6) and an average value of **2.71 gm/cc** has been taken for subsequent use in stability analysis.

2. **Bulk density** (saturated) of these rocks fluctuates between 2.70 and 2.78 gm/cc (Table 5.6). The average bulk density of **2.74 gm/cc** has been adopted.

3. **Porosity or Void Ratio** tends to vary between 2.04% - 4.50% (Table 5.6). Average porosity of biotite gneiss comes to **3.17%** and Void Index stands at **0.13%**.

Table 5.6 - Density & Porosity Parameters of Biotite Gneiss, Mangti Landslide

Type of Rock: **Biotite Gneiss**

Radius of the Specimen (r): 27mm; Length of the Specimen (l): 108mm

Type of Specimen: 54 mm Cylindrical Core Specimen

Sr. No.	Wt. of Dry Sample (M_d) (gm)	Wt. of Saturated Sample (M_{sat}) (gm)	Volume of specimen $(v = \pi r^2 l)$ (cc)	Dry Density (ρ_d) $\frac{M_d}{v}$ $\left(\frac{gm}{cc}\right)$	Bulk Density (ρ_{sat}) $\frac{M_{sat}}{v}$ $\left(\frac{gm}{cc}\right)$	% Porosity $\left(\frac{M_{sat} - M_d}{\rho_w v}\right) \times 100$
1	679.85	687.27	247.22	2.75	2.78	3.00
2	671.20	677.38	247.22	2.71	2.74	2.50
3	660.07	668.73	247.22	2.67	2.70	3.50
4	680.47	685.52	247.22	2.75	2.77	2.04
5	671.20	679.85	247.22	2.71	2.75	3.50
6	657.60	668.73	247.22	2.66	2.70	4.50

Determination of Water Absorption

For determining water absorption in all 04 samples were tested as per procedure of testing elaborated in Indian Standard Code of Practices IS : 1124 – 2003. The test results show water absorption of Biotite Gneiss rock samples ranging from **1.00%** to **1.725%** (Table 5.7) and an average value of **1.433%** is adopted.

Table 5.7: Water Absorption Test Results, Mangti Landslide

Type of Rock: **Biotite Gneiss**

Type of Specimen: Rock Pieces passing through 20mm IS Sieve & Retained on 10 mm IS Sieve.

Sr. No.	Wt. of Dry Sample (A)	Wt. of Saturated Samples (B)	% Water Absorption $= \left(\frac{B-A}{A}\right) \times 100$
1	502.5	510.0	1.492 %
2	492.5	501.0	1.725 %
3	500.0	505.0	1.000 %
4	495.0	502.5	1.515

STRENGTH PROPERTIES OF BIOTITE GNEISS ROCK AT MANGTI LANDSLIDE

Generally three kinds of stresses are considered in studying the stress resistivity of rock (Krynine and Judd, 1957): Compressive stresses, which tend to decrease the volume of the material; Shear stresses, which tend to move one part of a specimen with respect to the other or make it flow; and Tensile Stresses, which tend to produce cracks and fissures in the material. Accordingly the rock may have a compressive strength and a shearing strength; the tensile strength of the rocks is very negligible. Thus an attempt was made to evaluate these strength properties of Biotite Gneiss rock occurring at the Mangti Landslide. Ensuing paragraphs elaborate the results obtained from the tests carried on samples in laboratory.

Unconfined Compressive Strength (UCS)

The Unconfined Compressive Strength (UCS) denotes the stress required to break a loaded unconfined sample. UCS test was carried out under two different conditions i.e., in dry and saturated condition. In both conditions core samples (03 Nos. in each category) were tested along and perpendicular to the foliation planes in accordance with the IS : 9143 - 1996. Thereby, in all 12 samples were tested to determine UCS. Obtained UCS results under various conditions are listed in Table 5.8. The average adopted UCS values for biotite gneiss are **53.1 MPa** (along foliation), **82.2 MPa** (perpendicular to foliation), **25.26 MPa** (Saturated – along foliation) and **46.56 MPa** (Saturated – perpendicular to foliation). Thus it shows that the compressive strength depends on the direction of acting compressive stress with the direction of foliation plane and that the highest compressive strength is obtained when the compressive stress is normal to the foliation. Also saturation decreases the compressive strength of the rock and points towards a direct relationship with percent sorption and compressive strength; *As percent sorption increases, the compressive strength decreases.*

Table 5.8 - Uni-axial Compressive Strength Parameters of Biotite Gneiss

S.No.	Failure Load (P) (tonne)	Area of Contact Surface $A = \pi r^2$ Where ($r = 27mm$) (cm^2)	Uni-axial Compressive Strength $\sigma_c = \frac{P}{A}$ Mpa $\left(1Mpa = 100 \,{}^{t}\!/_{cm^2}\right)$
Loading Along the Foliation Plane (Dry Sample)			
1	11	22.91	48
2	12	22.91	52.37
3	13.5	22.91	58.92
Loading Across the Foliation Plane (Dry Sample)			
1	18	22.91	78.56
2	16	22.91	69.83
3	22.5	22.91	98.21
Loading Along the Foliation Plane (Bulk Sample)			
1	5.5	22.91	24
2	5.5	22.91	24
3	6.3	22.91	27.49
Loading Across the Foliation Plane (Bulk Sample)			
1	9	22.91	39.28
2	10.5	22.91	45.83
3	12.5	22.91	54.56

Determination of Strength of Rock Material in Triaxial Compression

The compressive and shear stresses in rock and any other material act simultaneously, and if a failure in a compressed rock occurs, this is generally a result of their common action. The triaxial test duplicates this behaviour of a material under combined compression and shear stress action. Five core samples of Biotite Gneiss rock from the Mangti Landslide were tested under triaxial compression for determining 'c' and 'Ø' values as per IS 13047 : 1996. It may be observed from the results (Table 5.9) that on increase of lateral restraint the ultimate bearing load of the test specimens also increased proportionately. The resultant values for cohesion **'c'** and angle of internal friction **'Ø'** were **8.40 MPa** and **44.44°** respectively. The triaxial data plot is given as Figure 5.3.

Table 5.9 - Tri-axial Strength Parameters of Biotite Gneiss, Mangti Landslide

No. of Samples Tested: 05 Nos.

Core Length-10.8 cms; Core Diameter-05.35 cms.

L/D-2.035 Core Head Area-22.45 cm^2

Sr. No.	Horizontal Stress σ_3 in MPa	Breaking Load in KN	Ultimate Bearing Load in KN	Failure Stress σ_1 in MPa
1.	**2**	117.6	117.6	**52.4**
2.	**4**	156.7	156.7	**69.8**
3.	**6**	225.4	225.4	**100.43**
4.	**8**	274.3	274.3	**122.2**
5.	**10**	294.0	294.0	**131.04**

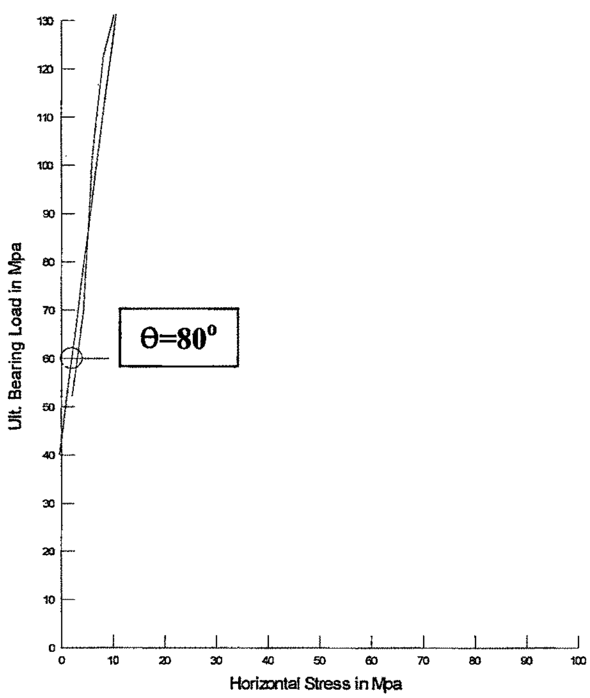

Figure 5.3 - Plot of Horizontal Stress σ_3 Vs. Ultimate Bearing Load σ_1 (Vertical Stress)

Calculations

$$\phi = \sin^{-1}\left(\frac{m-1}{m+1}\right) \quad \& \quad c = b.\left(\frac{1-\sin\phi}{2.\cos\phi}\right)$$

Where,

$m = \tan\theta$ & $b = Intercept$

$$m = \tan 80^0$$
$$b = 40MPa$$

Here, $\phi = \sin^{-1}\left(\frac{5.67-1}{5.67+1}\right) = \sin^{-1}(0.7001) = 44.44^0$

$$c = 40.\left(\frac{1-0.7001}{2.\cos 44.44^0}\right) = 8.40MPa$$

Tensile Strength (Brazilian Test)

Tensile strength is a measure of the tensile stress at which a material fractures or yields. To determine this, 03 samples each were subjected to tensile stress along the foliation plane and perpendicular to the direction of foliation plane in compliance to the Brazilian Test Method mentioned in Indian Standards Code IS 10082 : 2001. The obtained Tensile Strength along foliation plane shows range between **5.456** and **7.856 MPa**. Whereas, perpendicular to the direction of foliation plane it ranges between **11.13** and **11.79 MPa** (Table 5.10). Thus average values adopted for Tensile Strength are **6.4 MPa** and **11. 57 MPa** respectively.

Table 5.10 - Tensile Strength (Brazilian Test) Parameters of Biotite Gneiss

S.No.	Failure Load (P)	Diameter of Specimen(d)	Thickness of specimen(t)	Tensile Strength $\sigma_t = \frac{2p}{\pi dt}$ Mpa
Loading Along the Foliation Plane				
1	18	54mm	27mm	7.856
2	12.5	54mm	27mm	5.456
3	13.5	54mm	27mm	5.892
Loading Across the Foliation Plane				
1	27	54mm	27mm	11.79
2	27	54mm	27mm	11.79
3	25.5	54mm	27mm	11.13

An overall summary giving results of various geo-mechanical properties of biotite gneiss is given in Table 5.11.

Table 5.11 - Summary of Physico-Mechanical Properties of Biotite Gneiss, Mangti Landslide

Parameters	Values
Bulk Density	**2.74** $gms/_{cc}$
Dry Density	**2.71** $gms/_{cc}$
Absorption of Water Content	**1.433 %**
Bulk Specific Gravity	**1.13** $gms/_{cc}$
Porosity	**3.17 %**
UCS (Loading Along The Foliation Plane)	**53.1** Mpa
UCS (Loading Perpendicular to Foliation Plane)	**82.2** Mpa
UCS (Loading Along The Foliation Plane-Saturated)	**25.16** Mpa
UCS (Loading Perpendicular to Foliation Plane) (Saturated)	**46.56** Mpa
Tensile Strength (Along The Foliation Plane)	**6.4** Mpa
Tensile Strength (Perpendicular to Foliation Plane)	**11.57** Mpa
Cohesion	**8.40** Mpa
Internal Angle of Friction	**44.44°**
m-Values	**0.112**
s-values	**1**
Shear Strength Mpa	**13.54** Mpa