CHAPTER 4

FIELD AND LABORATORY DATA COLLECTION

4.1 Selection of Sites

Landslides are a common phenomenon in the hills. In India, the Himalayan Region is highly susceptible to landslide hazards. Among the three broad regions, the Lesser Himalayan region has seen an increase in landslide incidents due to huge population pressure, hydrology, climate and local topography. The Lesser Himalayan range in the Indian state of Himachal Pradesh and Uttarakhand possesses several lithologies. Among these, carbonate lithologies like Limestone, Dolomite and Dolomitic limestone are widely spread. Table 4.1 shows ten locations, spreading across the Lesser Himalayan range in the Indian state of Uttarakhand and Himachal Pradesh, having carbonate lithologies.

Table 4.1: Locations of carbonate lithologies in the study area

No.	Location	Lithological formation	No.	Location	Lithological formation
1	Bonga, Mandwa, Uttarkashi, Dichli and Bhailura (Uttarakhand)	Shyalna (Limestone and Dolomite)	**6**	Shillai, Sataun and Parara (Himachal Pradesh)	Krol (Dolomite and Limestone)
2	Luhri (Himachal Pradesh)	Larji (Dolomite and Limestone)	**7**	Bilaspur, Shimla (Himachal Pradesh)	Shali (Limestone)
3	Jonk, Shivpuri and Kaudiyala (along NH 58 Rishikesh to Devprayag) (Uttarakhand)	Krol (Limestone)	**8**	Nainital and Almora (Uttarakhand)	Krol (Dolomitic Limestone and Limestone)
4	Pipalkoti, Gopeshwar, Nandprayag and Langsi (Uttarakhand)	Pipalkoti and Deoban (Dolomite and Limestone)	**9**	Loharkhet, Munsiyari, Bageshwar and Pithoragarh (Uttarakhand)	Deoban, Tejam and Mandhali (Dolomite and Limestone)
5	Dharamkot, (Himachal Pradesh)	Shali (Limestone)	**10**	Dehradun, Mussoorie and Tehri (Uttarakhand)	Krol (Dolomite and Limestone)

Based on the field visits, ten sites have been selected from each identified location having residual soil cover. These ten sites have been used for detailed in-situ investigations and sample collection. The GPS locations of the 10 selected sites from 10 identified locations have been marked on Google Map using handheld GPS apparatus (Fig. 4.1).

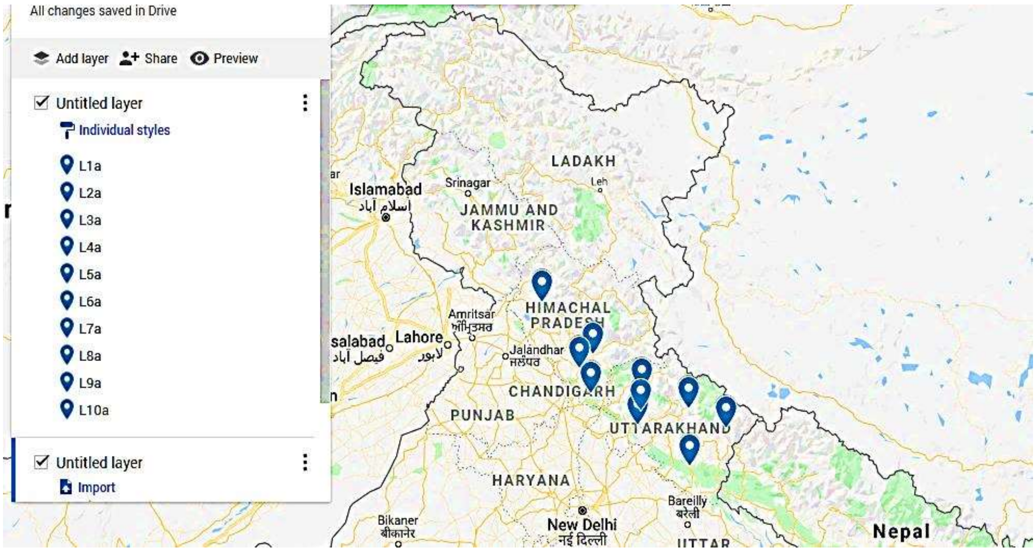


Fig. 4.1. GPS Coordinates of the selected sites

4.2 Field Investigation and Data Collection

Field investigations included the gathering of topographical features of the slope and discontinuity data of the bedrock. Geological mapping has been carried out where outcrops are exposed to delineate the discontinuities. Lithological studies have been performed for the physical properties of the bedrock and determinations of orientation, distribution & joint filling materials of discontinuities at the outcrops. The topographical parameters of the slope have also been examined, including slope inclination and residual soil depth. Various equipment that has been used in the field survey include Measurement tape, Geological hammer, Brunton compass, Schmid hammer, and Handheld GPS (Fig. 4.2). During field mapping of the bedrock properties, the orientations of different discontinuities were measured using Brunton compass. The Uniaxial Compressive Strength (UCS) were measured using Schmidt Rebound Hammer as per the International Society for Rock Mechanics (ISRM) suggested methods (Bieniawski 1989; Brencich et al. 2013; Ulusay 2014). The data of discontinuity (joints) orientation and spacings, infilling material, seepage flow (if any), weathering grade and orientations of slopes with dip angles have been recorded. The details of in-situ investigations on the selected sites are given in Table 4.2. Representative disturbed and undisturbed soil and rock samples were collected from all the sites to perform laboratory tests.

Table 4.2: Detail field investigation and in-situ tests at various selected sites

Site	**Coordinate**	**Rock Weathering**	**Discontinuity (Dip amount/ direction)**	**Joint Spacing (m)**	**Joint Infill**	**Rebound Hammer R-Value**	**Soil Depth (m)**	**Slope Inclination and Profile**	**Remarks**
S1	30.8470^{0}N, 78.2799^{0}E	M	56°/N350° 85°/N085° 58°/N190°	0.3-0.8	S	26-36	1-3	45^0-60^0 CX	Water seepage due to presence of a stream
S2	31.3459^{0}N, 77.4407^{0}E	M-H	65°/N175° 80°/N245°	0.2-0.8	C	30-46	1-3	35^0-55^0 CX	Poor rock strength and road cutting
S3	30.1373^{0}N, 78.3938^{0}E	M	67°/N040° 69°/N142°	0.3-0.9	C	22-32	1.5-4	35^0-45^0 CE	Steep slope and creep
S4	30.3366^{0}N, 79.319^{0}E	L-M	87°/N230° 25°/N345° 80°/N340°	1-3	S	26-46	1-5	40^0-55^0 CX	Instability due to road cutting
S5	32.2452^{0}N, 76.3259^{0}E	L-M	58°/N026° 78°/N277	0.4-0.8	C	16-26	0.5-2	40^0-60^0 S	Mass wastage due to local faulting and intersecting joints
S6	30.6842^{0}N, 77.3582^{0}E	L	68°/N100° 55°/N250° 50°/N130°	0.2-0.5	C	14-24	5-10	40^0-60^0 CE	Water seepage and soluble activity
S7	31.1025^{0}N, 77.1203^{0}E	M-H	45°/N174° 72°/N215°	0.2-0.8	S	24-36	1-2.5	45^0-65^0 CX	Rock block (detached). Solution activities observed
S8	29.3941^{0}N, 79.4598^{0}E	L-M	27°/N180° 56°/N120° 75°/N242°	0.5-2	S	28-38	1-5	45^0-60^0 S	Steep slope and seepage flow
S9	30.0616^{0}N, 80.2318^{0}E	L	30°/N190° 62°/N110°	0.03-0.1	S	16-24	1-3	50^0-65^0 CX	Debris occurrence over bedrock
S10	30.3739^{0}N, 78.0582^{0}E	L	30°/N330° 76°/N220° 75°/N135°	0.01-0.05	S	16-24	1-3	40^0-50^0 CE	Low shear strength of the slope material

(Note: Rock Weathering: Low (L), Moderate (M), High (H); Joint Infill: Clay (C), Sand (S); Slope Profile: Straight (S), Convex (CX), Concave (CE))

Fig. 4.2. Various apparatus used during field investigation

The general slope condition at the selected locations is dry except at sites S1, S6 and S8, where ground and surface water flow have been observed. Weathering has substantially altered and broken down the upper part of the bedrock in many sites by chemical decomposition and physical disintegration. The advances of weathering into fresh outcrop are visible in a few sites. Inherent bonding in the parent rock has gradually deteriorated in the course of decomposition because cohesion bonding decreases as the material decomposes. Due to limited cohesion, the slopes are susceptible to erosion and landslide during and after heavy rainfall, as observed in the form of several small-scale localised debris flow in the sites (Fig. 4.3).

Fig. 4.3. Several localised debris flows observed during site investigations

A clear stratigraphy of the residual soil slopes was observed at many sites (Fig. 4.4). The thickness of the top residual soil layer varies significantly from place to place and depends on the slope elevation (steep slopes are mostly devoid of the top residual layer). The extent of weathering depends on various natural and anthropogenic factors and the intensity of slope erosion (including gullies). Field investigation of various failed slopes revealed that many a time, the slope movement is initiated by the sliding of the residual soil mass, which lies in contact with the low-friction bedding planes. These planes have been exposed in a few areas after the failure of the soil layer (Fig. 3.6 and Fig. 4.5).

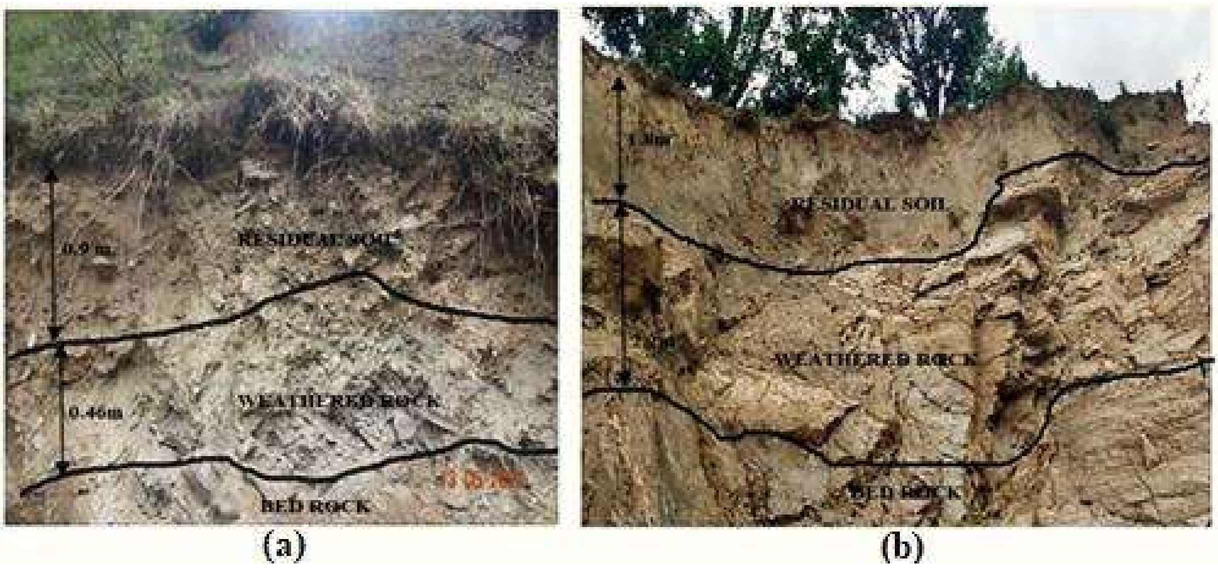


Fig. 4.4. Residual soil slope stratigraphy (a) Parashar, Himachal Pradesh (b) Sonprayag, Uttarakhand

Fig. 4.5. Failure of residual soil mass overlying a bedrock as observed in several locations in the study area

The outcrops in the study area are highly jointed and fractured at many sites. The rock mass study revealed the presence of two to three sets of joints at all the selected sites (Fig. 4.6). In a few sites (S6 and S7, as shown in Fig. 4.7), solution structures were noticed during seepage along the joints, making the joint aperture more than 5 mm. Joint surfaces were mostly smooth for all the selected sites. The major joint set at sites S1, S5, S6, S7 and S8 are almost parallel to the slope surface and are primarily responsible for the seepage on the slope during monsoon rainfall. A visible water scar was observed at the outcrop almost at all sites and especially at S1, S5, S6, S7 and S8. Splitting of the rock mass was easy using a geological hammer at most of the sites.

Fig. 4.6. Joint set pics at different sites

Fig. 4.7. Increased joint spacing (solution space) in calcareous rocks due to seepage

4.3 Laboratory Testing

Residual soil samples collected during field investigation were brought to the laboratory for index property identification (water content, grain size distribution, specific gravity), clay mineral identification, and strength parameter studies. Collection of undisturbed soil samples for strength tests was not possible due to considerable amounts of rock fragments and sand in the residual soil. Thus, a direct shear test was conducted in reconstituted/disturbed soil specimens. X-ray diffraction (XRD) analyses were also carried out to identify the constituent minerals of the collected soil samples. In the case of rock samples, specific gravity test, Poisson's ratio test, slake durability test, and strength parameters studies have been performed. All the laboratory tests have been performed as per the American Society for Testing Materials (ASTM) or ISRM standards.

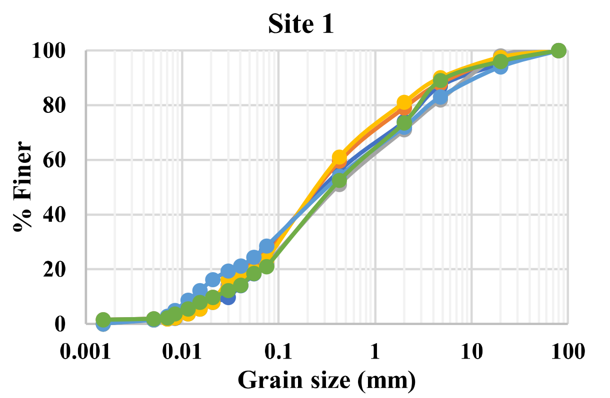
4.3.1 Grain Size Distribution of Residual Soil

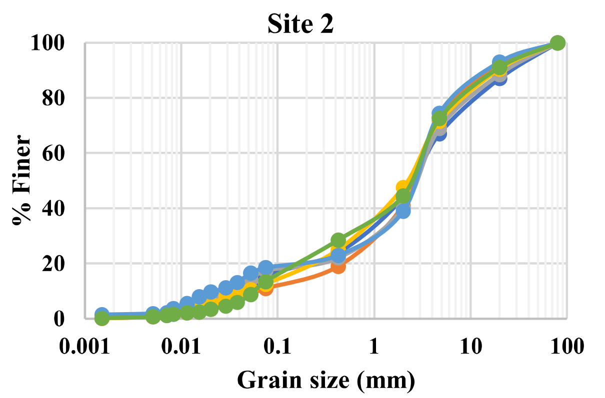
Grain size distribution analysis was performed using the shive shaker machine and hydrometer test as per ASTM guidelines D6913 and D7928, respectively (Fig. 4.8). Four sets of grain size distribution tests were carried out from the soil sample collected from each selected site. The average of the four tests was considered as the representative grain size distribution for one particular site. The results of the sieve and hydrometer analysis are shown in Fig. 4.9. Sieve analysis was performed using mesh sizes 80mm, 20mm, 4.75mm, 2mm, 0.425mm and 0.075mm. The classification for different size groups is assigned according to the Indian Standard Soil Classification System (ISSCS) size range given in Table 4.3.

Fig. 4.8. Grain size distribution analysis (a) Sieve Test and (b) Hydrometer Test

Table 4.3: Size range for soil according to ISSCS

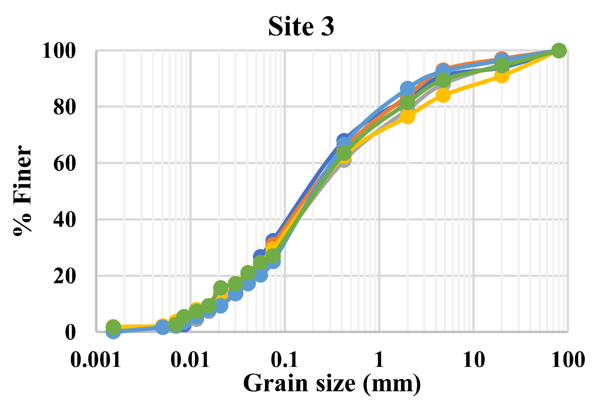
Size Range (mm)	Soil Type
<0.002	Clay
0.002-0.075	Silt
0.075-0.425	Fine Sand
0.425-2	Medium Sand
2-4.75	Coarse Sand
4.75-80	Gravel

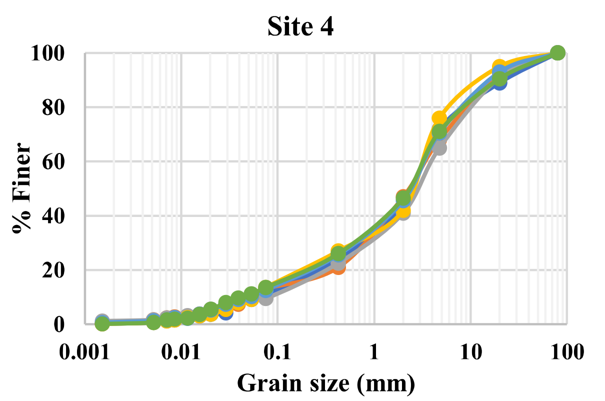




Cu= 26.66 D_{60}= 0.7 mm D_{10}= 0.03 mm
Cc= 0.267 D_{30}= 0.1 mm

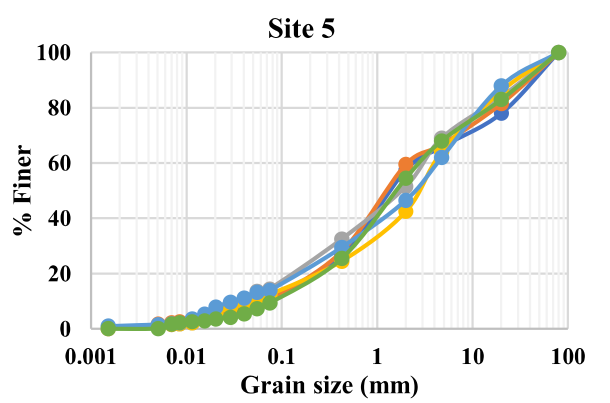
Cu= 70 D_{60}= 3.5 mm D_{10}= 0.05 mm
Cc= 4.62 D_{30}= 0.8 mm

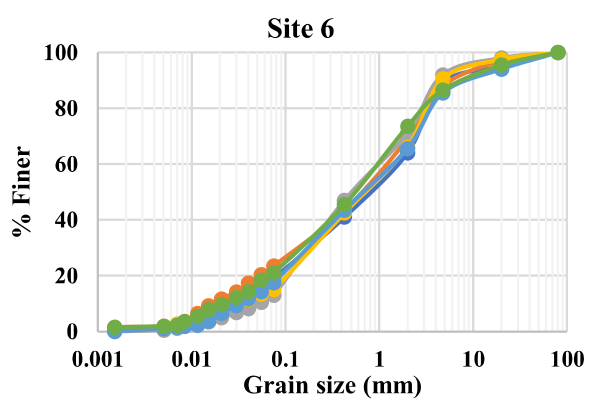




Cu= 15 D_{60}= 0.35 mm D_{10}= 0.02 mm
Cc= 0.82 D_{30}= 0.08 mm

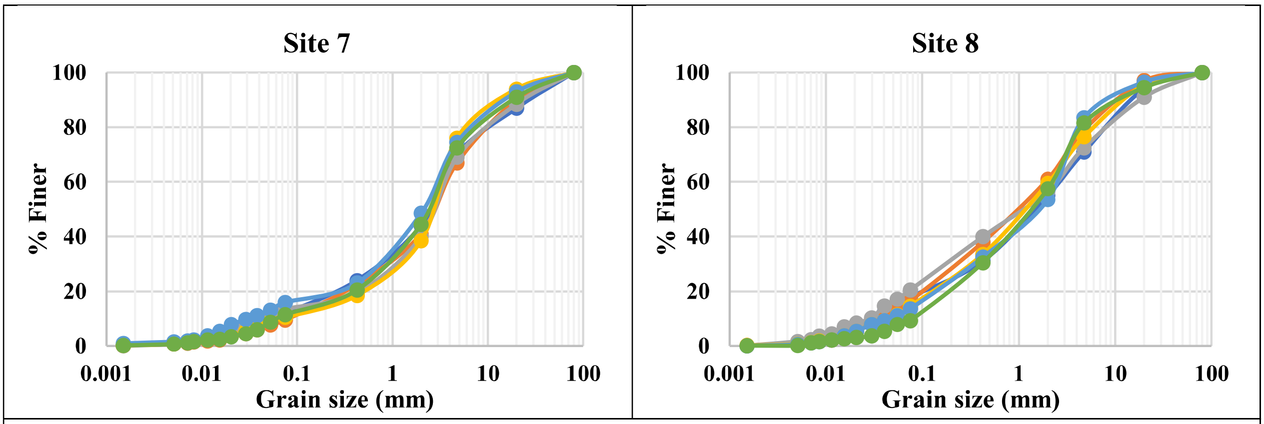
Cu= 70 D_{60}= 3.5 mm D_{10}= 0.05 mm
Cc= 3.65 D_{30}= 0.8 mm



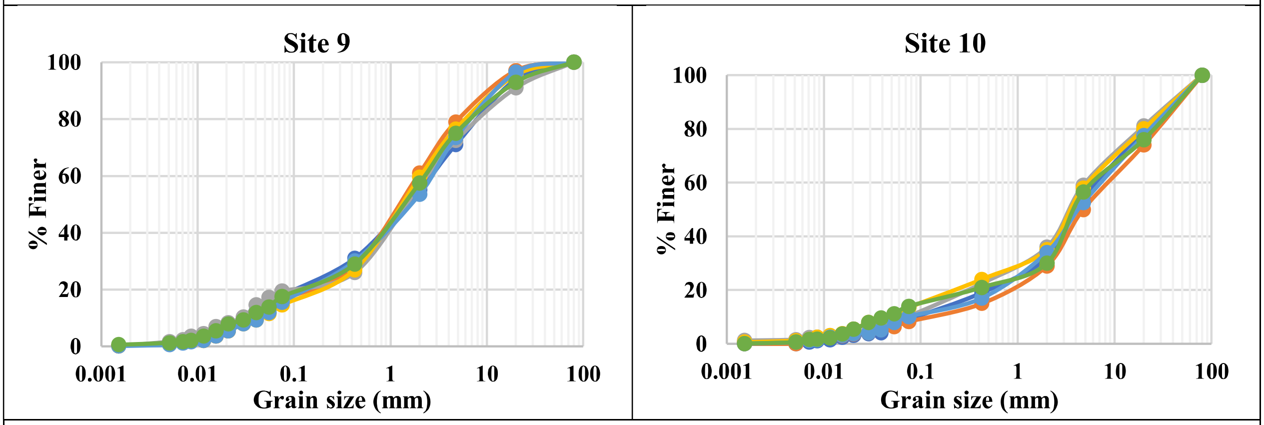


Cu= 30 D_{60}= 3.0 mm D_{10}= 0.04 mm
Cc= 0.3 D_{30}= 0.05 mm

Cu= 50 D_{60}= 1.5 mm D_{10}= 0.03 mm
Cc= 0.5 D_{30}= 0.2 mm



Site 7	Site 8
Cu= 50 D_{60}= 3.5 mm D_{10}= 0.06 mm Cc= 4.08 D_{30}= 1.0 mm	Cu= 28.57 D_{60}= 2.0 mm D_{10}= 0.04 mm Cc= 0.64 D_{30}= 0.02 mm



Site 9	Site 10
Cu= 66.67 D_{60}= 2.5 mm D_{10}= 0.04 mm Cc= 4.17 D_{30}= 0.4 mm	Cu= 85.71 D_{60}= 6.0 mm D_{10}= 0.07 mm Cc= 9.52 D_{30}= 2.0 mm

Fig. 4.9. Grain size distribution of the collected soil samples from various selected sites

Based on the grain size distribution analysis, the box and whisker plots were prepared (Fig. 4.10). It shows the variation in a particular size group of soil particles obtained from the grain size distribution analysis from all the sites. It was observed that the average clay fraction varies from 1.5 to 3.6%, average silt fraction varies mainly from 10.7 to 17.2%, average fine sand fraction varies mainly from 11.3 to 22.0%, average medium sand fraction varies mainly from 19.2 to 28.6%, average coarse sand fraction varies mainly from 11 to 23% and the average gravel fraction varies mainly from 4.0 to 12.3%. It was identified that the residual soil from the study area contains about an average of 82.6% of coarse aggregate (gravel and sand) and the rest 17.4% of fine aggregates (silt and clay), which are primarily well-graded (since C_u is >3 for all samples). The range of effective size (D_{10}) varies from 0.02 mm to 0.07 mm, indicating the sandy nature and high permeability of the soil. The unified soil classification system shows that the soil type of the selected sites is mainly silty clay of low plasticity (CL and CL-ML) with a considerable amount of sand and gravel.

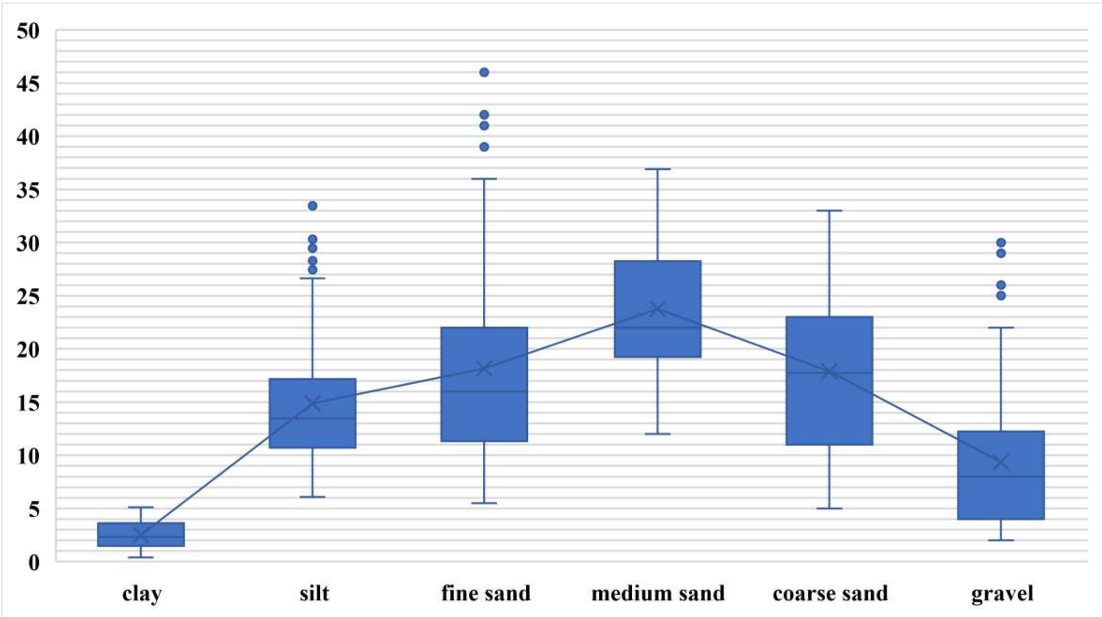


Fig. 4.10. Box and Whisker diagram showing the percentage variation in the particle size range of the residual soil samples

4.3.2 Natural Water Content of Residual Soil

The water content of the collected samples was tested as per ASTM D2216 (Fig 4.11). Three sets of natural water content tests were carried out from the soil sample collected from each site. The average of the three tests was considered as the representative natural water content for a particular site (Table 4.4).

Fig. 4.11. Oven drying for natural water content test

Table 4.4: Natural water content of the soil samples collected from various sites

Site	Sample 1	Sample 2	Sample 3	Avg	Site	Sample 1	Sample 2	Sample 3	Avg
1	20.3	19.6	19.0	19.63	6	27.4	28.5	28.1	28.00
2	7.4	5.6	6.8	6.60	7	14.3	15.9	13.8	14.67
3	13.9	12.8	14.6	13.77	8	22.9	24	22.6	23.17
4	11.7	13.1	12.1	12.30	9	14.6	12	11.6	12.73
5	16.1	13.4	13.8	14.43	10	11.7	13.1	12.1	12.30

Fig. 4.12 shows the box and whisker plot, which represents the variation in the natural water content of the soil. The results indicate that the natural water content varies between 6.6% to 28.0%. However, the average natural water content varies from 12.3 to 20.5% for most sites. The natural moisture content of the soil varies with season and plays a pivotal role in altering the density and strength of the material. The natural water content of the sites S1, S6 and S8 were relatively higher due to the seepage of water from the perennial streams.

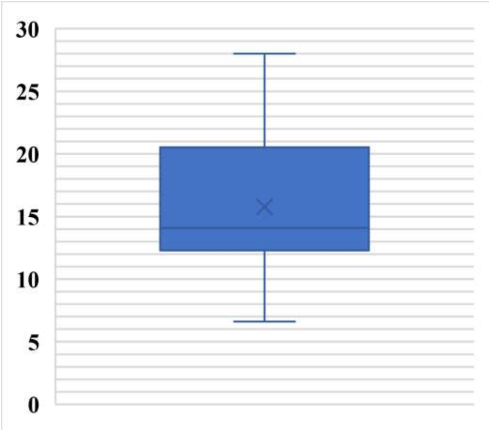


Fig. 4.12. Box and Whisker diagram showing the variation in natural water content (%) of the soil samples

4.3.3 Testing of Natural (Disturbed) and Remoulded Soil Samples

The laboratory study of the collected soil samples has been performed under two conditions. The first condition is in natural conditions, whereas the second condition represents the remoulded conditions (where both the percentage fine and water content of the soil are varied from 0% to 30%). The range of water content and percentage fine for the remoulded condition is selected based on laboratory investigation (twice the average value). The remoulded conditions will help in assessing the behaviour of the residual soil slopes at different time scales as both percentage fine (depend on weathering of bedrock) and water content varies over time.

4.3.3.1 Specific Gravity of Natural and Remoulded Residual Soil

To assess the solid fraction, the representative samples were passed through a 4.75mm sieve, which was analysed using a pycnometer for specific gravity (ASTM D854) (Fig. 4.13). Five soil samples were tested for specific gravity from each site. Out of the five samples, one is tested for the natural soil composition collected from the site (disturbed sample), and the rest four samples were remoulded soil with 0%, 10%, 20%, and 30% fine fraction (<0.075mm) in the tested soil sample (Table 4.5). It was observed that with an increase in the percentage of fine particles, the specific gravity increases. The specific gravity is directly proportional to density of the particles. Coarse grain soil has more void space which results into less density. While with the addition of fine particles, the void space gets filled up (without increasing the overall volume). This results in increase in the density of the soil which in turn increases the specific gravity.

Fig. 4.13. Specific gravity test

Table 4.5: Specific gravity test results for the natural (disturbed) and remoulded soil samples

Site	Natural Soil	Remoulded Soil				Site	Natural Soil	Remoulded Soil			
		0% Fine	10% Fine	20% Fine	30% Fine			0% Fine	10% Fine	20% Fine	30% Fine
1	2.72	2.64	2.71	2.74	2.78	6	2.68	2.63	2.67	2.69	2.72
2	2.63	2.59	2.64	2.66	2.69	7	2.70	2.67	2.70	2.72	2.76
3	2.70	2.66	2.70	2.72	2.77	8	2.67	2.62	2.65	2.68	2.70
4	2.55	2.47	2.61	2.64	2.68	9	2.65	2.61	2.66	2.68	2.71
5	2.68	2.65	2.67	2.69	2.71	10	2.68	2.65	2.68	2.70	2.73

4.3.3.2 Geo-mechanical Properties of Natural and Remoulded Residual Soil

A series of consolidated drained direct shear tests were performed under a fixed shearing rate of 1mm/min owing to the sandy and silty-sand nature of the samples (Lambe 1951; Mamo and Dey 2014). The samples were sheared under constant normal stress of 0.5, 1.0 and 1.5 kg/cm^2. Area corrections were performed before obtaining the stress-strain curve. The remoulded soil samples were prepared with 0%, 10%, 20% and 30% fine content and were tested for 0%, 10%, 20% and 30% water content. The young's modulus for the corresponding samples was obtained through the stress-strain curve obtained from the direct shear test. The void ratio of the samples was obtained using a cylindrical container test in the laboratory. Based on the void ratio (e), water content (w) and specific gravity (G), the degree of saturation (S) of the samples was obtained based on the formula given in Eqn 4.1 (Noorany 1984). The results of strength parameter analysis for natural (disturbed) soil samples are shown in Table 4.6. Results indicate that the cohesion, friction angle, young's modulus, void ratio and degree of saturation for the natural (disturbed) soil samples varies from 13-24 kPa, 33^0-42^0, 68-128 MPa, 0.45-0.80 and 31.5-96.2 % respectively. While the average value for each of these geotechnical parameters is 19.7 kPa, 37.6^0, 94.1 MPa, 0.61 and 67.81%, respectively.

$$S = \frac{wG}{e} * 100\% \tag{4.1}$$

Table 4.6: Geotechnical parameters of natural (disturbed) soil sample tested for natural water content

Site	**Cohesion (kPa)**	**Friction Angle (^{0})**	**Young's Modulus (MPa)**	**Void Ratio**	**Degree of Saturation (%)**
1	24	36	74	0.80	66.74
2	21	35	78	0.55	31.56
3	22	38	113	0.52	71.50
4	18	41	121	0.69	45.46
5	23	33	98	0.60	64.45
6	21	36	75	0.78	96.21
7	13	42	128	0.45	88.02
8	24	42	97	0.72	85.92
9	14	39	89	0.53	63.65
10	17	34	68	0.51	64.64

Table 4.7 shows the result of the direct shear test of remoulded residual soil samples under dry conditions, i.e., 0% water. Since testing was done under dry conditions, saturation was zero for all the remoulded conditions. In the case of 0% fine content, the friction angle, young's

modulus and void ratio for the remoulded soil samples varies from 40.3^0-43.9^0, 102-126 MPa and 0.62-0.80, respectively. While the average value for each of these geotechnical parameters is 42.9^0, 112.5 MPa and 0.69, respectively. As the testing was performed with 0% fine and dry condition, the cohesion of the remoulded samples was zero.

In the case of 10% fine content, the cohesion, friction angle, young's modulus and void ratio for the remoulded soil samples varies from 0-4 kPa, 40.1^0-43.2^0, 101-119 MPa and 0.55-0.69, respectively. While the average value for each of these geotechnical parameters is 2.35 kPa, 42.1^0, 108.3 MPa and 0.62, respectively.

In the case of 20% fine content, the cohesion, friction angle, young's modulus and void ratio for the remoulded soil samples varies from 3-5.8 kPa, 37.6^0-40.2^0, 92-108 MPa and 0.45-0.62, respectively. While the average value for each of these geotechnical parameters is 4.35 kPa, 39.1^0, 98.3 MPa and 0.54, respectively.

In the case of 30% fine content, the cohesion, friction angle, young's modulus and void ratio for the remoulded soil samples varies from 3.3-5.3 kPa, 35.3^0-37.6^0, 88-103 MPa and 0.40-0.57, respectively. While the average value for each of these geotechnical parameters is 4.24 kPa, 36.55^0, 94.3 MPa and 0.48, respectively.

It was observed that under dry conditions and with the increase in % fines, the value of cohesion increases till 20% fine content and then becomes almost constant. However, the values of friction angle, young's modulus, and void ratio decrease with an increase in % fines.

Table 4.7: Geotechnical parameters of remoulded residual soil sample tested for 0%WC

Site	Cohesion (kPa)	Friction Angle (^{0})	Young's Modulus (MPa)	Void Ratio	Degree of Saturation (%)
0% Fine Content					
1	0	43.6	123	0.63	0
2	0	40.3	112	0.68	0
3	0	43.4	106	0.62	0
4	0	43.9	102	0.65	0
5	0	42.4	124	0.69	0
6	0	43.6	107	0.72	0
7	0	41.7	126	0.78	0
8	0	43.3	107	0.71	0
9	0	43.6	112	0.80	0
10	0	43.2	106	0.68	0
10% Fine Content					
1	2.5	43.1	119	0.66	0
2	0.0	40.1	104	0.59	0
3	3.3	42.9	101	0.63	0
4	2.0	42.2	102	0.69	0
5	4.0	41.3	116	0.57	0
6	3.0	41.5	107	0.67	0
7	0.0	41.9	119	0.61	0
8	2.8	42.4	105	0.61	0
9	2.9	43.2	104	0.55	0
10	3.0	42.4	106	0.61	0
20% Fine Content					
1	4.5	40.1	108	0.51	0
2	4.0	37.6	94	0.61	0
3	4.8	39.7	93	0.48	0
4	3.3	39.7	92	0.45	0
5	3.0	38.2	106	0.46	0
6	3.3	38.4	97	0.51	0
7	3.8	38.7	108	0.54	0
8	5.5	39.2	95	0.62	0
9	5.5	40.2	94	0.62	0
10	5.8	39.2	96	0.58	0
30% Fine Content					
1	4.8	37.5	104	0.53	0
2	4.0	35.3	90	0.50	0
3	4.3	37.1	89	0.44	0
4	3.3	37.0	88	0.45	0
5	3.3	35.6	102	0.52	0
6	3.3	35.9	93	0.41	0
7	3.8	36.3	104	0.40	0
8	5.0	36.6	91	0.57	0
9	5.3	37.6	90	0.45	0
10	5.3	36.6	92	0.56	0

Table 4.8 shows the result of the direct shear test of remoulded residual soil samples under 10% water content. In the case of 0% fine content, the friction angle, young's modulus, void ratio and degree of saturation for the remoulded soil samples varies from 37.8^0-41.4^0, 94-118 MPa, 0.63-0.79 and 32.5-42.0%, respectively. While the average value for each of these geotechnical parameters is 39.74^0, 103.8 MPa, 0.73 and 35.74%, respectively. Since the testing was performed with 0% fine condition, the cohesion of the remoulded samples was zero.

In the case of 10% fine content, the cohesion, friction angle, young's modulus, void ratio and degree of saturation for the remoulded soil samples varies from 6-11 kPa, 34.9^0-38.3^0, 85-99 MPa, 0.56-0.72 and 36.9-47.3%, respectively. While the average value for each of these geotechnical parameters is 9 kPa, 36.78^0, 90.3 MPa, 0.65 and 41.29 %, respectively.

In the case of 20% fine content, the cohesion, friction angle, young's modulus, void ratio and degree of saturation for the remoulded soil samples varies from 19.5-35.5 kPa, 28.3^0-31.5^0, 86-102 MPa, 0.48-0.65 and 41.2-56.6%, respectively. While the average value for each of these geotechnical parameters is 27.5 kPa, 30.11^0, 92.6 MPa, 0.55 and 49.0 %, respectively.

In the case of 30% fine content, the cohesion, friction angle, young's modulus, void ratio and degree of saturation for the remoulded soil samples varies from 34-51 kPa, 26.5^0-29.6^0, 74-87 MPa, 0.40-0.60 and 46.0-67.5%, respectively. While the average value for each of these geotechnical parameters is 39.9 kPa, 28.28^0, 79.4 MPa, 0.50 and 55.2%, respectively.

It was observed that under 10% water content and with the increase in % fines, there is a significant increase in the value of cohesion and decrease in friction angle, young's modulus and void ratio. The average degree of saturation increased from 35% to 55%, with the increase in % fines from 0% to 30%.

Table 4.8: Geotechnical parameters of remoulded residual soil sample tested for 10%WC

Site	Cohesion (kPa)	Friction Angle (^{0})	Young's Modulus (MPa)	Void Ratio	Degree of Saturation (%)
0% Fine Content					
1	0	38.5	118	0.79	33.42
2	0	38.4	98	0.77	33.64
3	0	39.7	105	0.67	39.70
4	0	40.0	94	0.76	32.50
5	0	39.3	113	0.63	42.06
6	0	37.8	102	0.79	33.29
7	0	40.7	107	0.77	34.68
8	0	41.4	101	0.70	37.43
9	0	40.4	98	0.72	36.25
10	0	41.2	102	0.77	34.42
10% Fine Content					
1	9.0	35.7	99	0.62	43.71
2	10.5	35.6	86	0.64	41.25
3	7.5	36.8	86	0.63	42.86
4	9.5	37.0	85	0.56	46.61
5	7.5	36.4	98	0.69	38.70
6	6.0	34.9	89	0.71	37.61
7	8.0	37.6	99	0.57	47.37
8	10.0	38.3	87	0.66	40.15
9	11.0	37.4	86	0.72	36.94
10	11.0	38.1	88	0.71	37.75
20% Fine Content					
1	29.0	30.2	102	0.52	52.69
2	29.0	29.8	89	0.53	50.19
3	29.5	28.3	87	0.48	56.67
4	19.5	30.3	86	0.60	44.00
5	19.5	30.4	100	0.51	52.75
6	19.5	29.3	91	0.54	49.81
7	26.0	30.4	102	0.62	43.87
8	32.5	31.5	90	0.65	41.23
9	35.0	30.4	89	0.48	55.83
10	35.5	30.5	90	0.63	42.86
30% Fine Content					
1	38.5	28.4	87	0.54	51.48
2	38.5	28.0	76	0.42	64.05
3	42.5	26.5	75	0.60	46.17
4	34.0	28.5	74	0.49	54.69
5	34.0	28.6	86	0.54	50.19
6	34.0	27.6	78	0.52	52.31
7	34.0	28.5	87	0.41	67.32
8	46.5	29.6	77	0.40	67.50
9	46.0	28.5	76	0.59	45.93
10	51.0	28.6	78	0.53	51.51

Table 4.9 shows the result of the direct shear test of remoulded residual soil samples under 20% water content. In the case of 0% fine content, the friction angle, young's modulus, void ratio and degree of saturation for the remoulded soil samples varies from 34.2^0-37.6^0, 89-112 MPa, 0.66-0.80 and 63.3-79.10%, respectively. While the average value for each of these geotechnical parameters is 35.79^0, 98.6 MPa, 0.74 and 71.21%, respectively. Since the testing was performed with 0% fine condition, the cohesion of the remoulded samples was zero.

In the case of 10% fine content, the cohesion, friction angle, young's modulus, void ratio and degree of saturation for the remoulded soil samples varies from 3-5 kPa, 32.4^0-35.6^0, 80-94 MPa, 0.61-0.75 and 69.6-88.5% respectively. While the average value for each of these geotechnical parameters is 4.0 kPa, 33.87^0, 86.6 MPa, 0.66 and 81.81 %, respectively.

In the case of 20% fine content, the cohesion, friction angle, young's modulus, void ratio and degree of saturation for the remoulded soil samples varies from 11.5-20.5 kPa, 28.6^0-31.9^0, 71-84 MPa, 0.61-0.69 and 77.9-87.2% respectively. While the average value for each of these geotechnical parameters is 15.8 kPa, 30.5^0, 77.3 MPa, 0.64 and 84.47 %, respectively.

In the case of 30% fine content, the cohesion, friction angle, young's modulus, void ratio and degree of saturation for the remoulded soil samples varies from 25.5-43.5 kPa, 23.6^0-26.3^0, 59-68 MPa, 0.57-0.64 and 84.3-95.8% respectively. While the average value for each of these geotechnical parameters is 32.5 kPa, 25.1^0, 62 MPa, 0.60 and 90.7 %, respectively.

It was observed that under 20% water content and with the increase in % fines, there is a significant increase in the value of cohesion (however not that much as observed in case of 10% water content) and decrease in friction angle, young's modulus and void ratio. The average degree of saturation increased from 71% to 90%, with the increase in % fines from 0% to 30%.

Table 4.9: Geotechnical parameters of remoulded residual soil sample tested for 20%WC

Site	Cohesion (kPa)	Friction Angle (^{0})	Young's Modulus (MPa)	Void Ratio	Degree of Saturation (%)
0% Fine Content					
1	0	35.3	112	0.80	66.00
2	0	35.8	93	0.74	70.00
3	0	35.6	100	0.68	78.24
4	0	34.7	89	0.78	63.33
5	0	34.2	107	0.74	71.62
6	0	35.2	97	0.75	70.13
7	0	35.5	102	0.73	73.15
8	0	37.3	96	0.75	69.87
9	0	37.6	93	0.66	79.09
10	0	36.7	97	0.75	70.67
10% Fine Content					
1	4.0	33.4	94	0.62	87.42
2	4.0	33.8	84	0.61	86.56
3	4.5	33.7	82	0.61	88.52
4	3.0	32.8	80	0.75	69.60
5	3.5	32.4	94	0.74	72.16
6	3.5	33.3	85	0.64	83.44
7	3.5	33.6	94	0.63	85.71
8	4.5	35.3	84	0.63	84.13
9	5.0	35.6	84	0.67	79.40
10	5.0	34.8	85	0.66	81.21
20% Fine Content					
1	16.5	30.6	84	0.64	85.63
2	16.5	30.1	75	0.61	87.21
3	17.0	28.6	73	0.65	83.69
4	11.5	30.7	71	0.63	83.81
5	11.5	30.9	84	0.62	86.77
6	11.5	29.7	76	0.69	77.97
7	14.5	30.8	84	0.66	82.42
8	18.5	31.9	75	0.62	86.45
9	20.0	30.8	75	0.63	85.08
10	20.5	30.9	76	0.63	85.71
30% Fine Content					
1	32.0	25.2	68	0.59	94.24
2	32.0	24.8	59	0.63	85.40
3	33.5	23.6	59	0.59	93.90
4	28.5	25.4	59	0.62	86.45
5	28.5	25.5	67	0.57	95.09
6	28.0	24.5	61	0.57	95.44
7	25.5	25.4	68	0.64	86.25
8	39.5	26.3	60	0.64	84.38
9	34.5	25.3	59	0.60	90.33
10	43.5	25.5	60	0.57	95.79

Table 4.10 shows the result of the direct shear test of remoulded residual soil samples under 30% water content. In this case, the saturation level was 100% for all % fine content. In the case of 0% fine content, the friction angle, young's modulus and void ratio for the remoulded soil samples varies from 31.0^0-33.9^0, 73-87 MPa and 0.50-0.67, respectively. While the average value for each of these geotechnical parameters is 32.6^0, 79.6 MPa and 0.59, respectively. Since the testing was performed with 0% fine condition, the cohesion of the remoulded samples was zero.

In the case of 10% fine content, the cohesion, friction angle, young's modulus and void ratio for the remoulded soil samples varies from 0.5-5 kPa, 25.5^0-27.9^0, 56-65 MPa and 0.6-0.71, respectively. While the average value for each of these geotechnical parameters is 2.8 kPa, 27.1^0, 59.1 MPa and 0.65, respectively.

In the case of 20% fine content, the cohesion, friction angle, young's modulus and void ratio for the remoulded soil samples varies from 9.0-18.5 kPa, 23.6^0-26.3^0, 46-55 MPa and 0.60-0.69, respectively. While the average value for each of these geotechnical parameters is 13.0 kPa, 25.2^0, 50.4 MPa and 0.64, respectively.

In the case of 30% fine content, the cohesion, friction angle, young's modulus and void ratio for the remoulded soil samples varies from 17.0-30.1 kPa, 19.7^0-21.6^0, 45-52 MPa and 0.55-0.64, respectively. While the average value for each of these geotechnical parameters is 22.4 kPa, 20.9^0, 47.5 MPa and 0.58, respectively.

It was observed that under 30% water content and with the increase in % fines, there is an increase in the value of cohesion (however, not that much as observed in case of 10% and 20% water content but more than 0% water content) and decrease in friction angle, young's modulus and void ratio. The Friction angle was around 30-70% less than the friction angle under 0% water content.

Table 4.10: Geotechnical parameters of remoulded residual soil sample tested for 30%WC

Site	Cohesion (kPa)	Friction Angle (^{0})	Young's Modulus (MPa)	Void Ratio	Degree of Saturation (%)
0% Fine Content					
1	0	31.6	87	0.51	100
2	0	31.6	77	0.50	100
3	0	32.6	75	0.67	100
4	0	32.8	73	0.66	100
5	0	32.2	87	0.54	100
6	0	31.0	78	0.65	100
7	0	33.3	87	0.61	100
8	0	33.9	77	0.57	100
9	0	33.2	77	0.58	100
10	0	33.8	78	0.58	100
10% Fine Content					
1	3.0	27.7	65	0.64	100
2	0.5	26.0	56	0.60	100
3	3.5	26.8	56	0.62	100
4	2.5	27.1	56	0.71	100
5	5.0	26.6	64	0.67	100
6	4.0	25.5	58	0.63	100
7	2.0	27.5	65	0.63	100
8	2.5	27.9	57	0.66	100
9	2.5	27.3	56	0.65	100
10	2.5	27.9	57	0.69	100
20% Fine Content					
1	13.5	25.8	55	0.69	100
2	13.5	24.9	49	0.63	100
3	13.5	23.6	48	0.66	100
4	9.5	25.3	46	0.62	100
5	9.0	25.4	55	0.62	100
6	11.5	24.6	49	0.68	100
7	11.0	25.4	55	0.68	100
8	14.5	26.3	49	0.61	100
9	16.0	25.4	49	0.60	100
10	18.5	25.5	49	0.63	100
30% Fine Content					
1	22.5	21.50	48	0.58	100
2	21.0	20.15	45	0.59	100
3	23.5	20.75	47	0.59	100
4	20.0	21.00	48	0.60	100
5	20.5	20.65	51	0.55	100
6	19.5	19.75	47	0.58	100
7	17.0	21.35	52	0.56	100
8	26.0	21.65	46	0.57	100
9	24.0	21.15	45	0.64	100
10	30.0	21.60	46	0.59	100

4.3.3.3 Shear Strength Parameters of Residual Soil and Weathered Rock Interface

The shear strength properties of the joint interface between the residual soil layer and the weathered bedrock layer are also obtained from the direct shear test. The interface boundary between soil and rock is simulated in the direct shear test by placing fractured rocks in the lower half and the residual soil on the upper half of the shear box. The shearing plane is the soil-rock interface. Based on the test, the shear strength parameters were obtained for the natural (disturbed) samples (Table 4.11) and remoulded samples (Table 4.12 and Table 4.13). The results indicate that the cohesion values are very less and are present in the case when the % fines in the samples are 20% or more. Most of the shear strength is obtained from the friction angle. However, with the increase in % fines (from 0% to 30%) and water content (from 0% to 30%) of the soil, the friction angle of the soil rock interface reduces by almost 60-65%.

Table 4.11: Shear strength parameters of the residual soil and weathered rock interface for the natural (disturbed) residual soil samples

Site	Cohesion (kPa)	Friction Angle (^{0})	Site	Cohesion (kPa)	Friction Angle (^{0})
1	0.7	32.2	6	1.8	33.1
2	0.1	33.1	7	0.4	36.8
3	1.3	33.1	8	0.3	34.0
4	0.1	34.0	9	1.6	32.2
5	0.5	36.8	10	0.3	36.8

Table 4.12: Shear strength parameter Cohesion (kPa) of the remoulded residual soil and weathered rock interface

Site	0% WC 0% FC	0% WC 10% FC	0% WC 20% FC	0% WC 30% FC	10% WC 0% FC	10% WC 10% FC	10% WC 20% FC	10% WC 30% FC
1	0.0	0.0	0.0	0.6	0.0	0.0	1.0	1.6
2	0.0	0.0	0.0	0.1	0.0	0.0	0.1	0.2
3	0.0	0.0	0.0	1.2	0.0	0.0	1.8	3.0
4	0.0	0.0	0.0	0.1	0.0	0.0	0.1	0.2
5	0.0	0.0	0.0	0.5	0.0	0.0	0.7	1.2
6	0.0	0.0	0.0	0.9	0.0	0.0	1.3	2.2
7	0.0	0.0	0.0	1.2	0.0	0.0	1.8	3.0
8	0.0	0.0	0.0	0.7	0.0	0.0	1.1	1.8

9	0.0	0.0	0.0	1.5	0.0	0.0	2.3	3.8
10	0.0	0.0	0.0	0.5	0.0	0.0	0.7	1.2
Site	**20% WC 0% FC**	**20% WC 10% FC**	**20% WC 20% FC**	**20% WC 30% FC**	**30% WC 0% FC**	**30% WC 10% FC**	**30% WC 20% FC**	**30% WC 30% FC**
1	0.0	0.0	0.8	1.3	0.0	0.0	0.3	0.8
2	0.0	0.0	0.1	0.2	0.0	0.0	0.0	0.1
3	0.0	0.0	1.5	2.5	0.0	0.0	0.6	1.4
4	0.0	0.0	0.1	0.2	0.0	0.0	0.0	0.1
5	0.0	0.0	0.6	1.0	0.0	0.0	0.2	0.6
6	0.0	0.0	1.1	1.8	0.0	0.0	0.4	1.1
7	0.0	0.0	1.5	2.5	0.0	0.0	0.6	1.4
8	0.0	0.0	0.9	1.5	0.0	0.0	0.4	0.9
9	0.0	0.0	1.9	3.2	0.0	0.0	0.8	1.8
10	0.0	0.0	0.6	1.0	0.0	0.0	0.2	0.6

Table 4.13: Shear strength parameter Friction Angle (degrees) of the remoulded residual soil and weathered rock interface

Site	**0% WC 0% FC**	**0% WC 10% FC**	**0% WC 20% FC**	**0% WC 30% FC**	**10% WC 0% FC**	**10% WC 10% FC**	**10% WC 20% FC**	**10% WC 30% FC**
1	36.9	35.4	33.3	31.5	31.6	29.9	29.3	28.3
2	38.0	33.4	34.2	29.7	32.5	28.2	30.1	29.1
3	38.0	34.4	34.2	30.6	32.5	29.1	30.1	29.1
4	39.1	35.4	35.2	31.5	33.4	29.9	30.9	29.9
5	42.2	40.4	38.0	36.0	36.1	34.2	33.4	32.3
6	41.2	36.4	37.1	32.4	35.2	30.8	32.6	31.5
7	32.7	35.4	29.5	31.5	28.0	29.9	25.9	25.0
8	32.7	35.4	29.5	31.5	28.0	29.9	25.9	25.0
9	42.2	36.4	38.0	32.4	36.1	30.8	33.4	32.3
10	36.9	31.3	33.3	27.9	31.6	26.5	29.3	28.3
Site	**20% WC 0% FC**	**20% WC 10% FC**	**20% WC 20% FC**	**20% WC 30% FC**	**30% WC 0% FC**	**30% WC 10% FC**	**30% WC 20% FC**	**30% WC 30% FC**
1	28.4	28.4	25.4	24.9	25.6	24.9	24.2	22.4
2	29.2	26.8	26.2	25.6	26.3	25.6	24.9	23.0
3	29.2	27.6	26.2	25.6	26.3	25.6	24.9	23.0
4	30.1	28.4	26.9	26.3	27.0	26.3	25.5	23.7
5	32.5	32.5	29.1	28.4	29.2	28.4	27.6	25.6
6	31.7	29.2	28.3	27.7	28.5	27.7	26.9	24.9
7	25.2	28.4	22.5	22.0	22.7	22.0	21.4	19.8
8	25.2	28.4	22.5	22.0	22.7	22.0	21.4	19.8
9	32.5	29.2	29.1	28.4	29.2	28.4	27.6	25.6
10	28.4	25.2	25.4	24.9	25.6	24.9	24.2	22.4

4.3.3.4 High Resolution X-Ray Diffraction Analysis of Residual Soil

The mineralogical composition of the collected soil samples was studied by using the High-Resolution X-Ray Diffraction (HR-XRD) method. HR-XRD studies were made of randomly oriented specimens to examine mineral species in powder bulk samples and oriented specimens to facilitate clay-mineral identification. For the oriented specimens, the powder bulk samples were centrifuged after suspension in water to obtain clay fractions <2 μm in size (also known as XRD of clay minerals separated by a density method), and then slides were prepared for X-ray diffraction analysis. XRD patterns were recorded with a Rigaku SmartLab 9Kw Powder type diffractometer using graphite-monochromatized CuKα 40 kV and 40 mA (Fig 4.14). The diffractometer was calibrated using silicon as an external standard. XRD of the collected soil samples was done between 4^0 and 40^0 2θ steps size and 2 counting time per second. The powder method was used for the XRD analysis. All particles were crushed into a fine powder, and constituent minerals were identified using X-ray diffraction patterns. The ethylene glycol treatment method was used to confirm the clay minerals of low values of 2θ (less than 15^0). Soil samples collected from four major geological formations, namely Larji, Krol, Deoban and Shali were used for mineralogical analysis (Table 4.14).

Fig. 4.14. HR-XRD testing apparatus, Central Instrument Facility IIT (BHU)

Table 4.14: Mineralogical analysis of the residual soil samples using HR-XRD

Lithological Formations	Quartz (or Feldspar)	Muscovite	Illite	Smectite	Chlorite	Calcite (or Dolomite)
Larji	++++	-	+	+	+	+++++
Krol	++++	+	++	-	-	+++++
Deoban	++++	-	++	+	-	+++++
Shali	+++	-	+	+	+	+++++

(Note: Relative abundance: +++++ high ↔ + low; - absent)

From the X-ray diffraction patterns of the tested soil samples, it was observed that the main constituent minerals of the collected samples are quartz (or feldspar) and calcite/dolomite (depending on limestone/dolomite, respectively). While the soil samples have low to moderate concentrations of clay minerals like Smectite, Illite, and Chlorite. In general, the mineral Illite can be interpreted as the product of weathering of acidic igneous and high-grade metamorphic rocks. Chlorite is relatively unstable to weathering, and its presence indicated rapid corrosion on uplift or glaciation, particularly from low-grade metamorphic rocks. Smectite is formed under an impeded drainage environment. After analysing the samples, it was inferred that clay minerals could play an essential role in accelerating mass movement. Particularly in areas, which have the presence of sensitive clay-like Smectite. These clay minerals, whenever they come in contact with water, change their interlayer space, hence increasing in volume and, under dry conditions, again comes in their original state. This swelling and pinching character of the sensitive clays reduce the shear strength of the overburden, which gradually slumps down due to gravity. Another role of these minerals is that they sometimes act as a lubricant in the presence of water, thus helping the overburdened residual soil slide down the slope. The instantaneous deformation is due to the flexure of the plate-like clay particles, and slow deformation is due to the visco-plastic properties of clay minerals. When the percentage of fine particles are higher in the sample (as in the case of remoulded samples), the effect of clay minerals in the presence of water will affect the overall stability of the slope.

4.3.4 Geo-mechanical Properties of Intact Rock

A series of triaxial, Brazilian tensile, specific gravity and Poisson's ratio tests were performed according to ISRM suggested method using the rock samples collected from the sites (Fig 4.15). Before testing, the rock core samples were prepared from the samples collected from sites. The young's modulus for the corresponding samples was obtained through the stress-strain curve obtained from the triaxial test. The results of geo-mechanical analysis for rock samples are shown in Table 4.15.

Table 4.15: Geotechnical parameters of tested rock samples (dry)

Site	Cohesion (MPa)	Friction Angle (^{0})	Young's Modulus (GPa)	Tensile Strength (MPa)	Poisson's Ratio	Specific Gravity
1	14	22	40	8	0.30	2.64
2	17	23	51	13	0.26	2.74
3	16	32	47	12	0.29	2.72
4	13	39	59	9	0.27	2.86
5	11	30	42	7	0.32	2.67
6	15	32	46	9	0.29	2.69
7	17	45	59	11	0.27	2.85
8	19	32	52	10	0.28	2.84
9	13	27	44	9	0.31	2.63
10	15	37	49	10	0.30	2.73

Fig. 4.15. Laboratory tests on rock samples (a) Sample preparation (b) Poisson's ratio (c) Sample collected from sites (d) Specific gravity (e) Brazilian tensile and (f) Triaxial test

The results indicate that the cohesion, friction angle, young's modulus, tensile strength, Poisson's ratio and specific gravity of the rock samples varies from 11-19 MPa, 22^0-45^0, 40-59 GPa, 7-13 MPa, 0.26-0.32 and 2.63-2.86, respectively. While the average value of these parameters is 15 MPa, 31.9^0, 48.9 GPa, 9.8 MPa, 0.28 and 2.7, respectively. The coefficient of variation of these parameters is 15.7% for cohesion, 22.2% for friction angle, 18.6% for young's modulus, 16.9% for tensile strength, 17.3% for Poisson's ratio and 3.13% for specific gravity. Analysis of coefficient of variation indicates a greater level of dispersion of parameters (cohesion, friction angle, young's modulus, tensile strength, Poisson's ratio) around the mean while a much-concentrated result for specific gravity.

4.3.5 Degree of Weathering of the Outcrop Rock Mass

Chips and small rectangular blocks of the outcrop rock mass were taken to conduct the slake durability test as per ASTM D4644-16 (Fig 4.16). The Slake Durability Index (SDI) test is widely used to determine the disintegration characteristic of the weak rocks in order to assess the extent and susceptibleness of the bedrock towards weathering. This test is done to find out the resistance offered by a rock sample to weakening and disintegration when subjected to two standard cycles of drying and wetting in a slaking fluid, usually water. The results of SDI tests on the rock samples collected from various sites are given in Table 4.16.

Fig. 4.16. Slake Durability Test

Table 4.16: Weathering intensity of the outcrop rock mass as obtained through SDI test

Site	Retained Ratio after 1st cycle (%)	Retained Ratio after 2st cycle (%)	Description of Rock (Gamble (1971) and ASTM D4644-16)	Site	Retained Ratio after 1st cycle (%)	Retained Ratio after 2st cycle (%)	Description of Rock (Gamble (1971) and ASTM D4644-16)
1	97.39	94.45	Durable/Partly weathered	6	97.48	94.23	Durable/Partly weathered
2	97.57	94.76	Durable/Partly weathered	7	98.21	96.90	Very durable/Unweathered
3	97.23	94.21	Durable/Partly weathered	8	96.05	94.29	Durable/Partly weathered
4	98.54	96.72	Very durable/Unweathered	9	96.74	94.31	Durable/Partly weathered
5	96.82	93.85	Durable/Partly weathered	10	97.84	93.72	Durable/Partly weathered

The results obtained from SDI tests indicate that the rocks from the study area are mostly partly weathered due to their quartz-bearing nature (as indicated in Section 4.3.3.4) and the micritic nature of the Limestone lithologies. However, due to various natural (vegetation, hydrology and climate) and anthropogenic activities, the weathering rate has increased, marked by highly fractured outcrops, as seen in Fig. 4.6 and Fig. 4.7.

4.4 Geotechnical Data from Literature

A sizeable amount of data is required for carrying out a probabilistic analysis which will be able to model the heterogeneity in the engineering properties of the geomaterials with high reliability. Thus, to augment the data obtained through various in-situ and laboratory tests, a thorough study was performed to extract the mechanical and engineering data on soil and weathered rock mass from available literature on the study area. The engineering properties of the residual soil and weathered rock mass of the selected lithologies in the Lesser Himalayan region are compiled from various scholarly literature and summarised in Tables 4.17 and Table 4.18.

Table 4.17: Geotechnical properties of residual soil from the literature

Area	Bedrock Type	Young's Modulus (MPa)	Poisson Ratio	Cohesion (kPa)	Angle of Internal Friction (°)	Unit Weight (kN/m³)	Source
Nainital, Uttarakhand	Limestone	-	-	1.95	31	-	Gupta et al. (2016b)
NH58, Uttarakhand	Limestone	93-119	0.3	24.05-26.42	34.8-38.4	19.2-24.3	Siddque and Pradhan (2018)
Mussoorie, Uttarakhand	Dolomitic limestone	40-170	0.28	63-65	22-38	-	Gupta et al. (2016a)
Pipalkoti, Uttarakhand	Dolomitic limestone	100	0.3	7-20	33-43	18-20	Kanungo et al. (2013)
Pipalkoti, Uttarakhand	Dolomite	14-43	0.3-0.35	0-12	26-45	18-20	Pandit et al. (2016)
NH58, Uttarakhand	Dolomite	100	0.3	20	34	16	Falae et al. (2021)
Dharchula, Uttarakhand	Dolomitic Limestone	110	0.25	28.43-30.00	27.4	18	Solanki et al. (2019)

Table 4.18: Geotechnical properties of weathered rock mass from the literature

Area	UCS (MPa)	Young's Modulus (GPa)	Cohesion (MPa)	Angle of Internal Friction (°)	Tensile Strength (MPa)	Unit Weight (kN/m^3)	Poisson Ratio	Source
Kauri and Bakkal villages, Jammu Kashmir	27-264	30-96	22.5	29	-	26.26-30.28	0.13-0.33	Tiwari and Latha (2017)
Dehradun, Uttarakhand	25	20	0.18	33	-	26.70	0.3	Pal et al. (2012)
NH58 (Rishikesh to Devprayag), Uttarakhand	34-49 and 15-23	1-1.5	-	-	-	24.75-25.41	0.38	Siddque and Pradhan (2018)
Nainital, Uttarakhand	-	-	0.040	35	-	-	-	Sah et al. (2018)
Pipalkoti, Uttarakhand	-	71-73	0.118-0.150	20-25	11.8	25	0.27-0.3	Pandit et al. (2016)
Pipalkoti, Uttarakhand	79	9.5-15.3	1-2	47-56	0.2-0.38	27	0.3	Pain et al. (2014)
Pipalkoti, Uttarakhand	90	10-20	-	20-25	-	27	0.25-0.3	Kanungo et al. (2013)
Luhri, Himanchal Pradesh	-	45	19	32.8	14.5	26.7	-	Singh et al. (2015)
Pithoragarh, Uttarakhand	30-54	25	-	-	-	26-27	-	Kainthola et al. (2013)
Luhri, Himanchal Pradesh	95-138	-	16.5-21.5	29-38	11-17	26-27.9	-	Kainthola et al. (2013)
Mussoorie, Uttarakhand	25	20	-	-	-	-	0.25	Gupta et al. (2016a)
Bilaspur, Himachal Pradesh	-	40	-	-	-	27	-	Chakraborty et al. (2016)
Badrinath, Uttarakhand	-	6-22	0.76	50	0.113	26	0.3	Jana et al. (2016)
Jammu, Jammu and Kashmir	90-115	61-65	1.8-44	23-35	-	26.26-30.28	0.15	Latha and Garaga (2010)
Luhri, Himachal Pradesh	58-90	6-11	0.23-0.36	37-50	-	26	0.25	Mahanta et al. (2016)
Bilaspur, Himachal Pradesh	-	24.71-48.69	-	-	-	26-27	-	Mishra et al. (2018)
NH58, Uttarakhand	-	10	0.023	41-43	-	27	-	Falae et al. (2021)
Dharchula, Uttarakhand	100	50	-	-	-	22	0.15	Solanki et al. (2019)

4.5 Discussion

Several sites have been selected which possesses the residual soil formed from weathering of carbonate lithologies spreading across the Lesser Himalayan range in the Indian state of Uttarakhand and Himachal Pradesh. The in situ and laboratory tests allowed the reconstruction of the mechanical behaviour of the residual soil slope in the highly landslide vulnerable study area. Field investigation results show intensive weathering has substantially altered and broken down the upper part of the bedrock resulting in the occurrence of residual soil layer of varying thickness. The residual soil of the study area is primarily loose and sandy with an average of 82.6% of coarse aggregate (gravel and sand) and the rest 17.4% of fine aggregates (silt and clay), which are mostly poorly sorted or well graded. The clay fraction varies maximum up to 4%. The natural water content of the soil varies from 6.7% to 28.7%, with an average of 14.5%.

Various geotechnical parameters of the natural (disturbed) and remoulded soil samples were obtained, including specific gravity, cohesion, friction angle, young modulus and soil-rock interface strength parameters. Further, the specific gravity, void ratio and water content are used to calculate the degree of saturation. Remoulded samples were tested under varying percentage fines (0%, 10%, 20% and 30%) and water content (0%, 10%, 20% and 30%) in the soil. The range of water content and percentage fine for the remoulded condition is selected based on field and laboratory investigation (maximum two times the natural water and percentage fine content).

In the case of remoulded samples, it was observed that with the increase in water content and percentage fines, the cohesion increases. However, the increase is not uniform. The maximum increase in cohesion with the increase in percentage fines was observed in the case of 10% water content while the least for 0% water content. A significant reduction in friction angle was observed with increase in percentage fines and water content in the remoulded soil samples. XRD analysis on the collected soil samples indicates that the main constituent minerals are calcite/dolomite (depending on limestone/dolomite, respectively) and quartz (or feldspar). While the soil samples have low to moderate concentrations of clay minerals like Smectite, Illite, and Chlorite.

A series of triaxial, Brazilian tensile, specific gravity and Poisson's ratio tests were performed on the rock samples collected from the sites for obtaining the strength characteristics of the bedrock. The slake durability index test was also performed to determine the disintegration characteristic of the selected lithologies in order to assess the extent and susceptibleness of the bedrock towards weathering.

The obtained results from in-situ and laboratory tests for various geo-mechanical parameters of the slopes indicate the highly variable nature of the geomaterial. In order to augment the data collected from field and laboratory investigations, a detailed literature review was also performed to obtain the geo-mechanical data of the residual soil and the selected lithologies from the study area. The data obtained from field and laboratory tests coupled with the literature study have been used to perform numerical simulation using probabilistic analysis as given in the next chapter. The numerical simulation results will then be used to develop Landslide Hazard Charts to identify potential landslides.