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GEOLOGICAL FEASIBILITY FOR HIGHWAYS; A CASE STUDY FROM KHYBER REGION, PAKISTAN

Muhammad Osama[*1], Waqas Ahmad[2], Shabir Ahmad[3], Abdullah Nawaz[4], Ikram Ullah Shah[5]

[*1,2,3,4,5]*Department of Geology, University of Peshawar*

[*1]osamakhan@uop.edu.pk, [2]waqaskhan@uop.edu.pk, [3]shabir.dir1234@gmail.com, [4]abdullahbahadar789@gmai.com, [5]ikramullahshah056@gmail.com

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Corresponding Author: *
Muhammad Osama

Abstract
This Paper evaluates the geological feasibility for highway construction in the Khyber region through a comprehensive geotechnical investigation. The study includes drilling boreholes, excavating test pits, and conducting various field and laboratory tests. The results provide insights into the subsurface conditions, necessary for designing and constructing a safe and stable highway. The mountainous terrain of the Khyber region in Pakistan presents unique challenges for highway construction. This Paper evaluates the geological feasibility for highway construction in this region through a comprehensive geotechnical investigation. The study employs a multi-pronged approach, including drilling boreholes, excavating test pits, and conducting various field and laboratory tests on the collected soil and rock samples. The results provide crucial insights into the subsurface conditions, a critical factor for designing and constructing a safe and stable highway infrastructure. This Paper delves into the methodology employed, presents the detailed findings, analyses the data to assess geological feasibility, and offers recommendations for successful highway construction in the Khyber region.

1. Introduction

1.1 General Statement

Geology is a cornerstone of engineering projects, providing essential insights into the earth's materials and processes that affect construction and long-term stability. It encompasses the study of soil and rock properties, groundwater behaviour, and natural hazards, all of which are critical for designing and constructing safe, durable infrastructure. Engineering geology involves geotechnical investigations, such as drilling boreholes and conducting field tests, to gather data on subsurface conditions. This information is crucial for making informed decisions about site selection, foundation design, slope stability, and material sourcing. By understanding geological conditions, engineers can mitigate risks associated with landslides, soil liquefaction, and other geohazards, ensuring the safety and sustainability of projects(Christopher & McGuffey, 1997)

The geological feasibility of highway construction involves assessing the suitability of the terrain and subsurface conditions along the proposed route. This process includes evaluating soil and rock properties, groundwater levels, and the presence of geological hazards such as fault lines and landslide-prone areas. Geotechnical investigations, which typically involve drilling boreholes and excavating test pits, provide detailed information on the subsurface conditions. For example, in a case study from the Khyber region, geotechnical investigations were based on three rotary drilled boreholes, each 10m to 21m deep, and seven test

pits. These investigations revealed crucial data on the soil's shear strength, compressibility, and potential for settlement, informing the design of foundations and slope stabilization measures.
Moreover, understanding the geological context helps in identifying suitable construction materials, such as aggregates and gravel, and assessing their availability and quality. Groundwater studies are also essential, as they influence drainage design and the risk of water-related issues like erosion and soil liquefaction. In seismically active areas, seismic hazard assessments are conducted to design earthquake-resistant structures and avoid building near active fault lines. Ultimately, integrating geological studies into highway engineering ensures that projects are designed with a comprehensive understanding of the ground conditions, leading to safer, more resilient infrastructure (Wyllie & Mah, 2017).

1.2 Study Area

Our study area, lies on the western border of the District Khyber in Torkham area which is a part of the Khyber Pakhtunkhwa Province, lying along the northwestern boundary of Pakistan with Afghanistan. The study area occurs along the Torkham import and export corridor covering an area of approximately 2.5 km. This study covers the hilly part of the whole corridor, as shown in figure-1.1

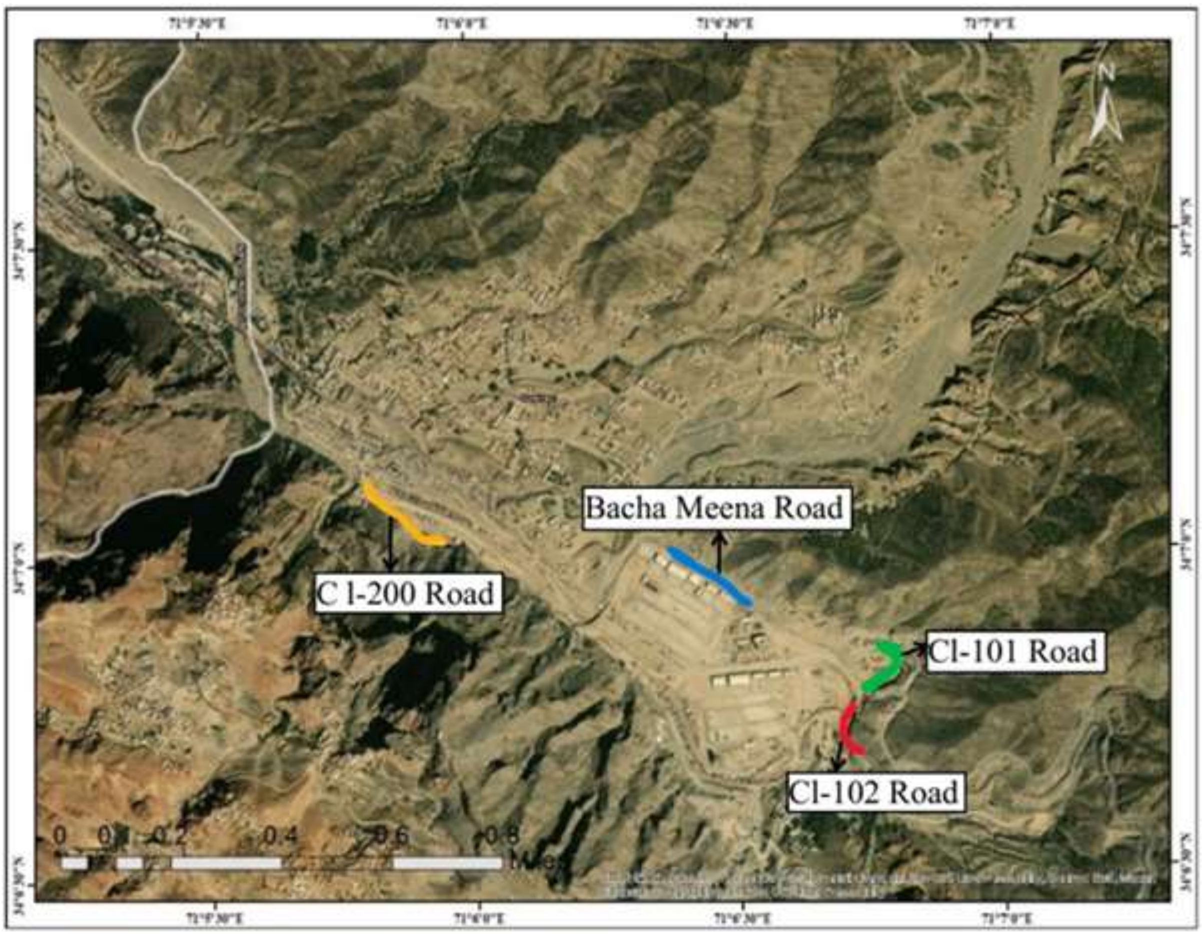

Figure 0.1 Google Earth Image of the Study Area

1.3 Aims And Objective

The objective of our study is to carryout geotechnical investigations and propose geotechnical recommendations for the highways at Torkham.

The objectives of this study can be summarized as under,

1. Drilling of three boreholes, each 10m to 21m deep, using straight rotary machine.
2. Excavation of seven (07) test pits
3. Standard Penetration Test/Cone Penetration Test at different intervals
4. Large scale/small scale Direct Shear Test
5. Point Load Test on Rock cores
6. Sieve Analysis
7. Moisture Content of soil/rock sample.
8. In-situ determination of Bulk Density/Dry Density of the soil.

1.4 Methodology

1.4.1 Field Mapping

Geological mapping has been carried out with the help of handheld GPS and various soil and rock units were delineated and marked on the base map. various rock units identified and marked on the base map. During mapping, a field assessment of the subsurface bed rock was also made keeping in view the geomorphology and orientation of the rock units. Representative sampling was also undertaken during the geological mapping for rock confirmation and onward laboratory testing.

1.4.2 Bore hole and Test Pits Details

Different locations of boreholes and tests pits were marked during geological study of the area of interest. Geotechnical investigation is based on a three (3) number of rotatory drilled boreholes each 10m to 21m deep and seven (7) number of test pits for the proposed geological study.

Table 0.1 Details of boreholes and Test Pits

Boreholes Details				
BH/Test Pit ID	**Location**	**No Boreholes/Test Pits**	**Depth of Borehole/Test Pit(m)**	**Borehole Coordinates**
1	Bacha Meena	1	10	34° 6'52.33"N, 71° 6'32.82"E
2	Entry-Exit Point (CL-101 + CL-102)	1	21	34° 6'45.58"N, 71° 6'41.36"E
3	CL-200	1	15	34° 7'4.82"N, 71° 5'50.66"E
Test Pits Details				
1	Bacha Meena	2	1.5	34° 6'51.83"N, 71° 6'33.56"E
2				34° 6'55.22"N, 71° 6'30.90"E
3	Exit Point (CL-101)	1	1.5	34° 6'47.53"N, 71° 6'48.17"E
4	Entry Point (CL-102)	1		34° 6'43.92"N, 71° 6'40.63"E
5	CL-200	3	1.5	34° 6'58.75"N, 71° 6'4.42"E

6				34° 7'4.19"N, 71° 5'51.91"E
7				34° 7'2.50"N, 71° 5'56.54"E

2 Regional Geology

2.1 Regional Geology

The formation of Himalayan Orogenic Belt in the early tertiary, was due to the head-on, N–S collision of India and amalgamated Cimmerian blocks, which extends over 2300 km from Afghanistan to Burma with 6100m average elevation(Pogue et al., 1992). Based on lithology and regional thrust faults, the Himalayan Orogenic Belt has been divided into the Greater Himalayan, Lesser Himalayan, and sub-Himalayan(DiPietro & Pogue, 2004). The rocks of Greater Himalayan are present between the Main Mantle Thrust (MMT) and Main Central Thrust (MCT). In Pakistan, the MCT is equivalent to the Panjal Khairabad Thrust fault (DiPietro & Pogue, 2004). These rocks are multiply metamorphosed and have been intruded by igneous rocks ranging in age from Cambrian to Tertiary(DiPietro & Pogue, 2004). The lesser Himalayan lies between Panjal-Khairabad Thrust in the north and Main Boundary Thrust in the south whereas Sub Himalayan lies between Main Boundary Thrust in the north and Salt Range Thrust in the south. The Khyber Ranges is a part of the Himalayan Crystalline Nappe and Thrust Belt which is an Intensely tectonized zone forms the Northwestern margin of the Indo-Pakistan crustal plate and Lies between the Panjal-Khairabad Thrust and Indus Suture Zone (MMT) (Kazmi & Jan, 1997). It extends westwards from the Nanga Parbat-Harmosh Massif up to Sarobi fault in Afghanistan. Its southern part is largely covered by the quaternary deposits of Peshawar and Haripur basins. North of Panjal-Khairabad Thrust, these basins are surrounded by low hill ranges comprised of Precambrian metasediments and a near complete sequence of fossiliferous Paleozoic and Early Mesozoic rocks. In the southern part this metasedimentary sequence has been affected by low grade metamorphism Northward near the Indus Suture Zone the tectonometamorphic setting changes from an essentially the Indus Suture Zone the tectonometamorphic setting changes from an essentially sedimentary fold and thrust belt to a metamorphic and magmatic terrain which is characterized by thick stacks of nappes, and mylonitised shear zones. In this complex fold and thrust belt three structural zones are quite evident. From north to south these includes the crystalline nappe zone, the Khyber-Hazara metasedimentary fold and thrust belt and the Peshawar Basin (Kazmi & Jan, 1997).

2.2. Khyber-Lower Hazara Metasedimentary Fold-and-Thrust Belt

This metasedimentary fold and thrust belt lie to the north of Panjal- Khairabad Thrust and extend eastward from the Khyber Pass region to Garhi Habibullah. To the northeast along the Hazara Kashmir Syntaxis, this belt is wedged in between the Panjal Thrust and the Salakot Shear Zone (Kazmi & Jan, 1997). The Mansoura Thrust which comprises a mylonitised shear zone, possibly an extension of the Salakot Shear Zone(Baig & Lawrence, 1987) separates the Khyber Hazara metasedimentary belt from the Hazara-Crystalline Nappe-Zone. The Khyber Hazara metasedimentary belt is largely composed of Precambrian to Early Mesozoic sediments (Kazmi & Jan, 1997). The Precambrian sequence is mainly comprised of slates and phyllites with subordinate quartzites and marbles which crop out in the southern part of the belt. The Precambrian section is largely in the form of the thrust blocks with variations in metamorphic grade at some places. The Khyber Hazara Meta sedimentary Fold and Thrust Belt has been intruded by dykes and sills and granitic rocks of which the extensive Ambala pluton is the most conspicuous. These intrusive rocks ranges in the age from Late Palaeozoic to Early Mesozoic. This metasedimentary belt is characterized by tight asymmetrical or isoclinal folds imbricated by several thrust faults(Kazmi & Jan, 1997).

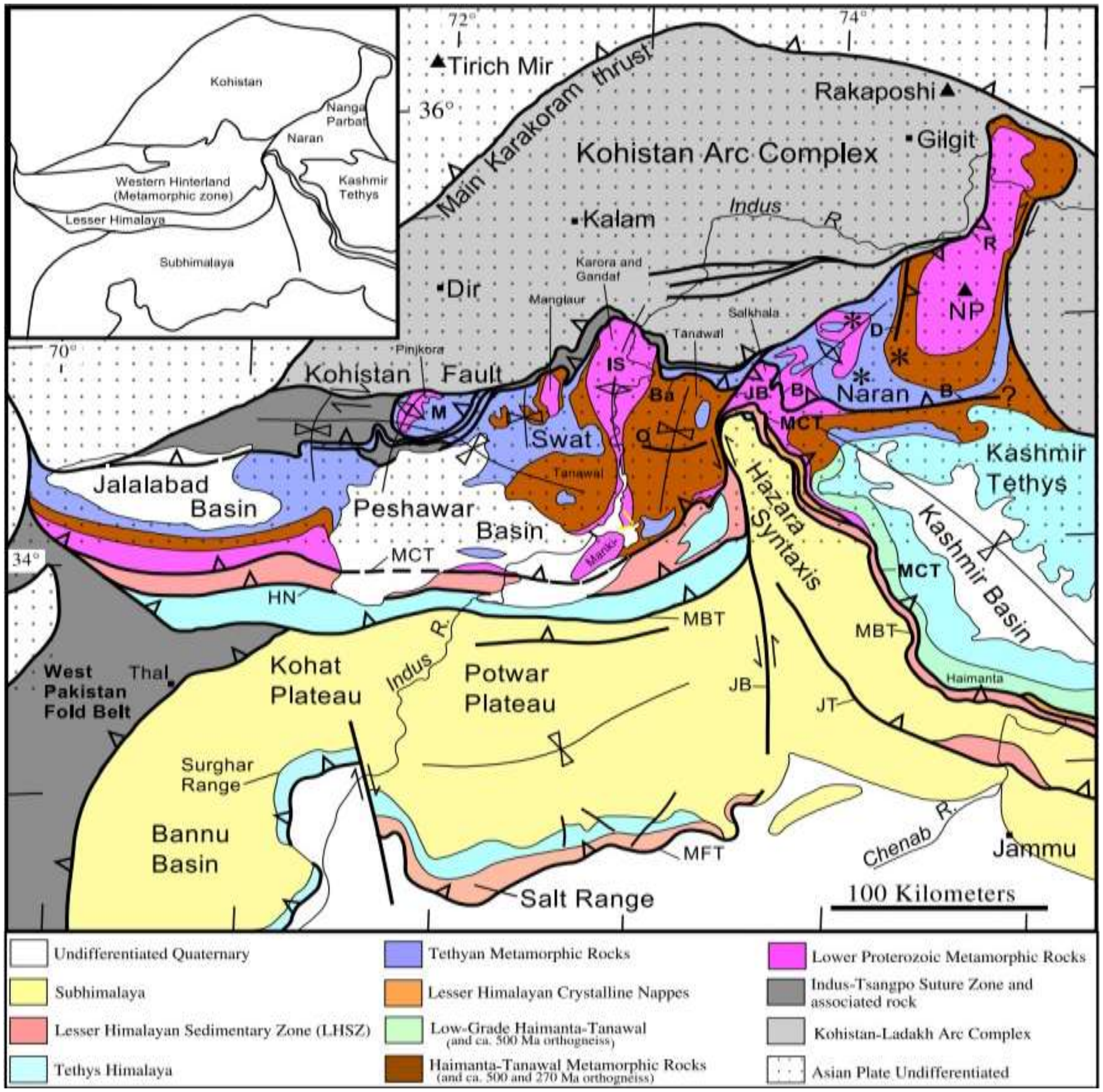

Figure 2.1: Tectonic map of the Western Himalaya showing major tectonic divisions (Source: DiPietro and Mouse, 2004).

2.2 Local Geology

The study area lies in Torkham Town of Khyber District of KPK near Pakistan-Afghanistan border. The geology of the District Khyber is quite complex; the field access to different parts of the areaand a general absence of chronologic markers adds to this complexity and a disagreement between different workers in the area ((Jan & Khan, 2015).The Khyber Pass area w\as divided into four units, in which the oldest is Landikotal Formation overlain by Shagai Formation which in turn is overlain by Ali Masjid Formation and the topmost Khyber Limestone (Shah et al, 1980).

Figure 2.1 Showing the stratigraphic sequence of Khyber Area in accordance with Shah, 1977.

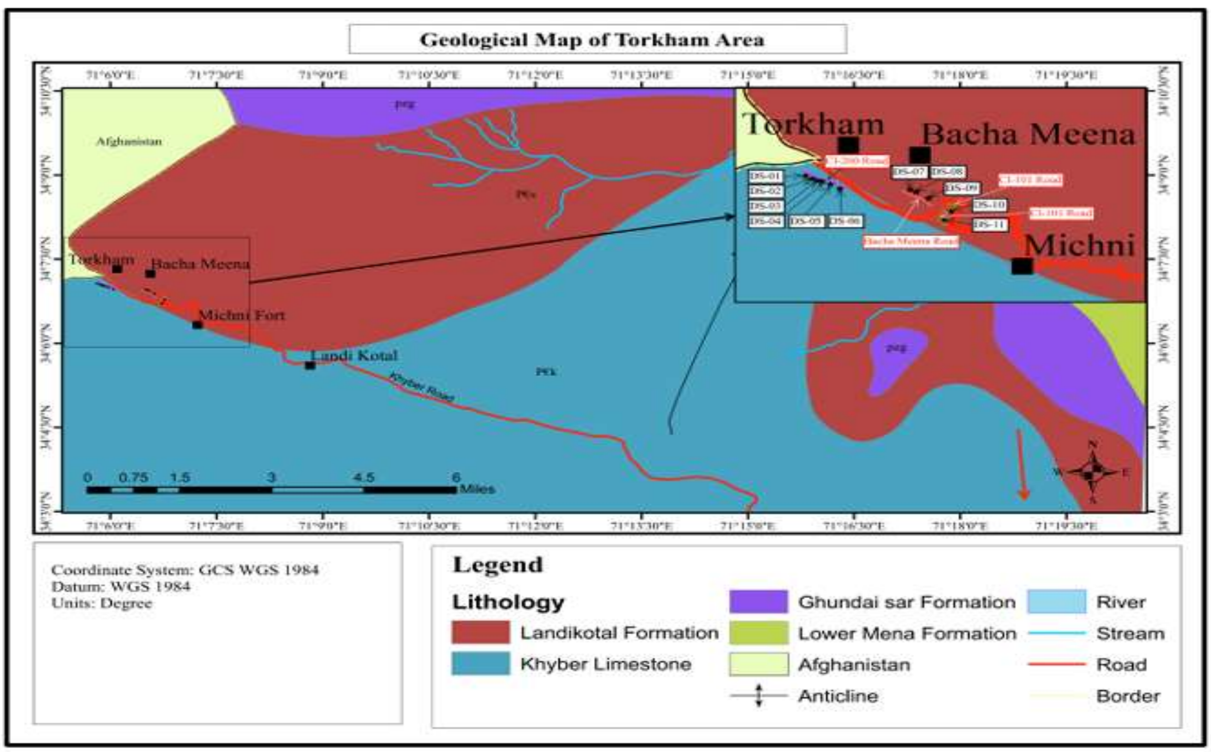

Figure 2.2 Geological map of study Area showing all the roads and DS locations (Map Redrawn from Khan and Aslam, 1989)

The Landikotal Formation is an assemblage of slate, phyllite, limestone, quartzite, basic rocks and at places dolomite. In the presence of such varied lithologies the entire unit has been named as the Landikotal Formation (Shah.S, 1969), with the section exposed to the north and east of Landikotal village being considered as the type locality. At type locality it consists of greenish-grey to yellowish grey phyllite and fine-grained slate with abundant basic igneous dykes and sills. The weathered surfaces of the slate have produced yellowish, greyish-green colour and in some places pencil structures. Minor amount of fossiliferous, medium bedded limestone layers interbedded with the slate are present with grey, weathering to light yellowish grey in colour.(Shah.S, 1969) The Landikotal formation extends deep into Afghanistan, where German geologists mapped part of the formation as Loger Formation . They reported the presence of Ordovician conodonts in the Loger Formation and assigned the age from Late Ordovician to middle Ludlow The Landikotal slates and Khyber Limestone at places are intruded by doleritic dykes. The exposure of the Khyber Limestone is widespread, extending from near Bara Fort to Loe Shilman in a north-south direction and from Ghund Garh to Bazar and perhaps beyond in an east-west direction. In the Study area, slates of Landikotal Formation are present in the north dipping northward having alignments of three road CL101, CL102 and Bacha Meena, while Khyber Limestone is present on the southern side also dipping northward along the CL 200 road alignment. The contact of the slate and limestone is documented to be faulted (Shah et al., 1980)

3 Geological Investigation of Tests Pits

3.1 Field Activities

3.1.1 Drilling

The purpose of the drilling investigation is to evaluate the subsurface conditions within the proposed stretches, CL-200, CL-101, CL-102 and Bacha Meena and to determine the engineering characteristics of the underlying soils and rock. The drilling consists of advancing 3 boreholes to a total depth of 10m to 21m below the existing ground surface. Additionally, at seven different locations, the test pits are to be excavated for the collection of bulk samples for conducting small/large scale direct shear tests Straight Rotary Drilling technique was used in the project. The boreholes were drilled by rotating a bit, and cuttings was removed by continuous circulation of the drilling fluid (bentonite) as bit penetrates the formation. The bentonite slurry is also used as a lining medium for bore material as well as to avoid erosion that may be encounter by using only water. The bit was attached to the lower end of a drill pipe, which transmits the rotating action from the rig to the bit. In the direct rotary system, drilling fluid containing bentonite was pumped down through the drill pipe and out through a jets in a bit, the fluid then flowed upward in the annular space between the hole and the drill pipe, carrying the cuttings and suspension to the surface. At the surface, the fluid was channelled into a settling pit. Pits were excavated for temporary use during drilling and backfilled after completion of the well. The collapse of borehole, particularly in granular/overburden soil, was prevented using thick bentonite slurry, cement-bentonite grouting or casing the borehole. Grouting was carried out during drilling for soils that were prone to collapse. Grouting materials consisted of slurry of cement and bentonite, pumped under high pressure into the borehole and remained it in undisturbed form for 24hrs so that to make a perfect bond with soils. The injected grout eventually formed a gel within the injected voids. The purpose of pressure grouting of a soil mass was to improve the strength and durability of the grouted mass and to reduce chances of collapse phenomena. Casing is also used where borehole walls collapsed. Casing is a pipe that is assembled and inserted into a drilled section of a borehole. Casing is set inside the drilled borehole to protect it from falling loose strata. The lower portion (and sometimes the entirety) is typically held in place. It provides a strong upper foundation to allow use of high-density drilling fluid to continue drilling deeper. The hole drilled for each casing must be large enough to accommodate the casing to be placed inside it. Tricone bit and clay bit were used where clay and sand were encountered during drilling. Where soft to medium hard rock was encountered tungsten carbide bit was used. For hard rock, diamond drill bit was used.

3.1.2 Standard penetration Test and Soil Sampling

Samples were obtained using an unlined standard sampler, called split spoon sampler consisting of a 2-inch outer dia., and 1.4-inch inner dia. The split spoon sampler is a tube split into two equal halves lengthwise. The two halves are locked together during the sampling activities and released to retrieve the samples. At bottom end of the sampler sits a driving shoe. This is what cuts into the soil and provides the sample that goes up into the tube. At the other end of the tube is a coupling that allows it to connect to the drilling rod. Once a sample is taken, the operator removes the ends from the tube. This allows the tube to "split" open. Representative samples can be taken for laboratory analysis. One of the unique advantages of this kind of sampling equipment is that it can be used on most drilling rigs. The drilling rigs will drill down to a certain level and, at that point, the drilling team will install the split spoon and insert it down to the bottom of the hole. Split spoon sampler was driven into the soil using a 140-pound (63.5kg) hammer free-falling a vertical distance of 30inches (750mm). The number of hammer blows required to drive the SPT sampler in three six-inch segments, blow count, were recorded during sampling. The combined blow count for the final two six inch is referred to as the SPT N-value. Sampling procedures employed in the field were generally consistent with those described in ASTM D1586. Samples were collected at regular intervals. Soil collected inside the split spoon sampler was visually classified in the field, placed in sealed plastic bags and stored for future reference and laboratory testing. Where hard gravels were encountered the split spoon sampler is replaced by attaching a solid cone with the drilling rods as permitted by BS 1377-90.

3.1.3 Test Pits

Test pits at various locations are excavated in the project site for conducting in-situ field density test and for collection of bulk samples to be transported to laboratory for testing for direct shear testing. The details of the tests pits are given in Table 1.

3.1.4 Field Dry Density Test

This test method is used for the determination of in-place density of soil using sand cone apparatus in accordance with ASTM D-1556. The soil used for testing should have sufficient cohesion and particle attraction to maintain stable sides on a small hole or excavation. This test method is not suitable for soils having organic, saturated or highly plastic soils that would deform or compress during the excavation of hole. A test hole is hand excavated and all materials excavated have been saved in a container. The hole is filled with free-flowing sand of known density and the volume of hole is determined. Bulk density of soil is determined by dividing excavated material on total volume of hole. Dry density is then calculated using bulk density and water content ratio.

The results of field dry density tests are shown in Table below: The results of field dry density tests are shown in Table below:

Table 3.1 Field Density test

Test Pit ID	Natural Moisture Content %	Bulk Unit weight (gm/cc)	Dry Unit weight (gm/cc)
1	3.1	1.51	1.47
2	2.6	1.8	1.75
3	1.8	1.86	1.83
4	2.1	1.73	1.70
5	2.0	2.01	1.97
6	1.4	1.94	1.91
7	1.7	2.59	2.55

The field dry density tests indicate generally favorable subgrade conditions along the highway alignment. Natural moisture content varies between 1.4% and 3.1%, reflecting relatively dry soil conditions that are advantageous for load-bearing performance and reduced deformation under traffic loads. Lower moisture contents observed in Test Pits 3 and 6 suggest improved compaction potential, whereas Test Pit 1, with comparatively higher moisture content, may exhibit greater sensitivity to settlement and therefore requires careful compaction control. Bulk unit weight values range from 1.51 to 2.59 g/cm³, with higher values indicating denser and more stable soil conditions; Test Pit 7 exhibits the highest bulk density, implying superior resistance to compression under highway loading, while the lower density recorded at Test Pit 1 suggests a greater potential for settlement if not adequately compacted. Similarly, dry unit weight values (1.47–2.55 g/cm³) confirm that soils at Test Pit 7 are well compacted and mechanically competent, whereas lower values at Test Pit 1 may necessitate additional compaction effort or stabilization measures. Owing to the predominantly gravelly nature of the subsurface materials, undisturbed sampling was not feasible; therefore, representative bulk soil samples (~50 kg) were collected and transported to the laboratory for moisture content determination and direct shear testing.

3.2 Laboratory Testing

Rock and soil samples collected during the field exploration were transported to SSCP geotechnical laboratory for testing. The purpose of laboratory testing was to;

Following laboratory tests were/will be conducted.

- Point Load Test on Rock Cores (ASTM D –5731)
- Uniaxial Compression Test (ASTM D –7012)
- Small/large scale Direct Shear Test (ASTM D 3080)
- Sieve Analysis (ASTM D 422)
- Moisture content Determination (ASTM D –6913)

3.2.1 Point Load Test

The Point load test is an index test by which the rock is classified according to the strength. The test can be used to estimate other characteristics of intact rocks with which it correlates, such as uniaxial compressive and tensile strength. Point Load Test is conducted on four rock cores obtained from Borehole No 1 at Bacha Meena and the tests results are tabulated in the table below.

Table 3.2 Point Loads Test Results on Rock Cores (Slate) from BH-1 at Bacha Meena

Core Information from Field					Lab. point Test Data					
Sample ID	Core Depth		Core Length (cm)	Core Dia(cm)	Core Dia (cm)	Core Length (cm)	Failure Load (N)		Is50 (Mpa)	Estimated UCS(Mpa)
	From	TO					Axial	Diametral		
C1	3.5	5	10.5	4.6	4.6	9.5	0	8829	4.02	48.2
C2	5	6.5	7	4.6	4.6	6.5	0	6867	3.13	37.5
C3	5	6.5	8.2	4.6	4.6	7	0	7848	3.57	42.9
C4	5	6.5	11.5	4.6	4.6	10.4	0	7848	3.57	42.9

Table 3.3 Point Loads Test Results on Rock Cores (Limestone) from Landslide Area

Sample ID	Core Dia(cm)	Core Length (cm)	Failure Load (N)		Is50 (MPa)	estimated UCS (MPa)
			Axial	Diametral		
C1a	43.6	40.6	13734	-	5.95	93.4
C1b	43.6	50.6	-	6867	3.4	53.3
C2a	43.6	41.2	12753	-	5.46	85.8
C2b	43.6	74.8	-	6867	3.4	53.3
C3a	43.6	39	10791	-	4.76	74.7
C3b	43.5	34.7	11772	-	5.77	90.6
C3c	43.5	75.3	-	8829	4.38	68.8

Point load testing of slate cores obtained from depths ranging between 3.5 m and 6.5 m indicates consistent mechanical behavior within the investigated subsurface interval. The tested cores exhibit a uniform diameter of 4.6 cm, while core lengths vary from 6.5 to 11.5 cm, reflecting minor structural variability in the rock mass. Diametral failure loads range from 6,867 N to 8,829 N, suggesting moderate to high tensile strength of the slate. The calculated point load strength index (Is_{50}) values vary between 3.13 MPa and 4.02 MPa, classifying the slate as moderately strong. Corresponding uniaxial compressive strength (UCS) values, estimated from Is_{50}, range from 37.5 MPa to 48.2 MPa, exceeding the minimum strength requirements commonly considered suitable for highway subgrade and embankment support. These results indicate that the slate lithology possesses adequate load-bearing capacity to sustain highway construction under anticipated traffic loads.

3.2.2 Uniaxial Compressive Strength

Four rock samples are obtained from CL-200. Uniaxial compression tests are then performed on rock cores extracted from the rock samples. The tests were conducted perpendicular to the foliations of the rock samples. This test is carried out according to the ASTM D 7012.

Table 3.4 Uniaxial Compression Test Results on rock cores obtained from rock samples at CL 200.

SR N0	Failure Load(kN)	Diameter (mm)	Length (mm)	Surface Area(mm)	UCS (Mpa)	UCS Corrected (50mm dia) (Mpa)
1	143	46.01	43.1	1662.6	86.01	84.73
2	134	45.93	42.94	1656.8	80.88	79.65
3	126	46.17	43.35	1674.2	75.26	74.19
4	110	46.11	43.26	1669.9	65.87	64.92

The uniaxial compressive strength (UCS) values of the tested rock samples range from 65.87 to 86.01 MPa, which reduce slightly to 64.92–84.73 MPa after correction to a standard core diameter of 50 mm. Although some variability is observed among the samples, with the highest strength recorded in Sample 1 and the lowest in Sample 4, all values fall within the moderately strong to strong rock classification (50–100 MPa). This strength range indicates that the rock mass possesses sufficient load-bearing capacity to support highway subgrades, embankments, and associated structural elements.

3.2.3 Large Scale Direct Shear Test

Large scale direct shear test performed on soil samples in accordance with ASTM D-3080. The test performed by deforming a specimen slowly until it reached to peak shear resistance and no resistance encountered beyond. Three specimens were tested, each under a different normal load, to determine the effects upon shear resistance and displacement. From graphical representation of normal vs shear resistance, shear strength parameters like cohesion and angle of internal friction determined. Similarly, shear resistance vs horizontal displacement as well as vertical displacement vs horizontal has been obtained.

The RQD of the rock core samples almost approach to zero. In such cases the rock can be treated as soil mass. A general guideline of friction angles for different types of rocks are given below.

Table 3.5 Shear Strength Parameters obtained from Large Scale Direct Shear Test

Sample ID	Depth	Cohesion (kpa)	Friction Angle (Degree)
PIT 02	1	8	46.4
PIT 05	1	16	50.3
PIT 07	1	10	44.3

The large-scale direct shear test results indicate favorable shear behavior of the tested materials, with cohesion values of 8–16 kPa and friction angles ranging from 44.3° to 50.3°. The moderate cohesion and relatively high friction angles reflect effective interparticle bonding and strong resistance to sliding, confirming the suitability of the materials for highway subgrades, embankments, and slope stabilization.

3.2.4 Grain Size Analysis Test

Sieve Analysis was performed by means of wet sieving in accordance with ASTM D 422. In sieve analysis, the mass of soil retained on each sieve determined and expressed as a percentage of the total mass of the sample. The particle size plotted on a logarithmic scale so that two soils having the same degree of uniformity are represented by curves of the distribution plot. From this it is possible to determine whether the soil consists of predominantly gravels, sand, silt or clay. For those soil which passes through sieve#200, the hydrometer analysis performed in accordance with ASTM D-7928. Hydrometer analysis is based on the principle of sedimentation of soil grains in water. A soil specimen is prepared and dispersed in water and placed for 24 hours. After 24hrs, the specimen is then dispersed and started stopwatch for taking continuous reading on specified time interval in accordance to ASTM.

Table 3.6 Sieve Analysis Results

PIT ID	% Gravel	% Sand	% Fines	Cu=D60/D10	Cc=(D30)2/(D10*D60)	USCS Classification
1	51	48	1	13.6	1.4	GW Well Graded Gravels with sand
2	46	54	0	11.8	0.66	SP Poorly-Graded Sand with Gravel
3	54	43	3	10	1.48	GW Well Graded Gravels with sand
4	61	39	0	18	2.13	GW Well Graded Gravels with sand

5	51	49	0	9	1.53	GW Well Graded Gravels with sand
6	48	52	0	27.2	0.37	SP Poorly-Graded Sand with Gravel
7	36	31	33	-	-	GM Poor Graded Silty Gravel with Sand

The above table shows the classification of different soil samples based on their **grain size distribution** and **USCS (Unified Soil Classification System)**, which is crucial for highway construction. This classification helps in determining the suitability of soils for subgrades, embankments, and foundation design.(Hoek & Brown, 1997). Grain-size analysis and USCS classification show that the subsurface materials are predominantly granular and suitable for highway construction. Most samples are classified as GW (well-graded gravel with sand), exhibiting favorable gradation (Cu and Cc), good compaction characteristics, high strength, and effective drainage. Samples classified as SP indicate poor gradation and may require stabilization, while the GM material, containing higher fines, may exhibit reduced drainage and strength under wet conditions. Overall, the gradation characteristics confirm the general suitability of the materials for highway subgrades and pavement support, with localized improvement required where poor gradation or excessive fines occur.

Results And Discussions

- The evaluation of test pits indicates varying soil suitability for highway subgrade construction. **Test Pits 6 and 7** exhibit high density with low moisture content, making them ideal for providing strong support under pavement. Conversely, **Test Pit 1** has the highest moisture and lowest unit weights, suggesting a higher risk of settlement, which may necessitate additional stabilization methods like compaction or the use of additives such as lime or cement. **Test Pits 2, 3, 4, and 5** show moderate properties, indicating that while they can perform adequately under standard practices, moisture control will be essential to prevent strength loss during construction.
- The Uniaxial Compressive Strength (UCS) values, ranging from 64.92 MPa to 84.73 MPa, demonstrate that the rock possesses a robust load-bearing capacity, making it highly suitable for roadbeds, embankments, and subgrade preparation. This strength enables it to effectively support both static and dynamic loads from heavy traffic.
- The strong rock characteristics also help mitigate the risks of deformation, subsidence, or failure over time, which is vital for maintaining the longevity and safety of highway infrastructure. Furthermore, while there is some variation in failure loads and UCS values among the samples, these differences are minimal, indicating a consistent mechanical property across the tested area. This consistency simplifies the design process for highway construction, ensuring reliable performance.
- The results from the Large-Scale Direct Shear Test reveal that the material exhibits favourable shear strength parameters, particularly regarding the friction angle, which is essential for slope stability and preventing shear failure. Although the cohesion values are somewhat lower, they remain adequate for most highway applications
- It is important to monitor areas with reduced cohesion to ensure long-term stability.

Overall, the material is well-suited for highway construction, especially concerning slope and embankment stability. However, reinforcement measures should be implemented in regions with lower cohesion to mitigate the risk of shear failure.

- The shear strength parameters of the tested materials are advantageous for highway construction, particularly regarding the friction angle, which reflects a strong capacity to resist shear failure. The moderate cohesion values indicate that specific areas, especially those with lower cohesion like PIT 02, might necessitate stabilization measures. Overall, the materials are appropriate for use in embankments, subgrades, and foundation support. However, sections with weaker characteristics may require additional reinforcement to maintain long-term stability.

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