FEASIBILITY LEVEL GEOLOGICAL AND GEOTECHNICAL ASSESSMENT OF BAZ ALI DAM AT LOWER KURRAM, KHYBER PAKHTUNKHWA, PAKISTAN.

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A thesis submitted to Quaid-i-Azam University Islamabad in partial fulfillment of requirement for the degree of Master of Philosophy in Geology.

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DEDICATION

Dedicated to my loving parents, who determined the pathway and destination for me and make things possible for me, my loving brothers and sisters whose prays and support enabled me to complete my studies.

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First of all, I bow my head to Almighty **Allah,** who bestowed me with the strength and capability to successfully complete this research work and made me proud of my tiny efforts without his help as would not have happened.

Humblest thanks to our beloved and Holy Prophet, **Hazrat Muhammad (PBUH),** with the core of heart, which is the undying fountain of knowledge and guidance for the whole mankind.

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Tausif Javed

ABSTRACT

A comprehensive research work was accompanied to the intended area i.e. District Lower Kurram, Baz Ali dam. The key subject of the research work was to study the area geologically and geotechnically and to guarantee that the area is feasible for dam construction. From fieldwork and over-all laboratory tests on different samples taken from outcrop, boreholes and test pits were analyzed and thus it was confirmed that the study area is comprised of major rocks units of mélange zone of Late-Cretaceous age thrusting over the east trending folded rocks of Jurassic, Cretaceous and Paleocene rocks. According to the geomorphology and geological conditions and economic reason, the dam has been designed as an earth dam with a clay core that can help the dam to be less permeable that is the reason the dam foundation is comprised of a sequence of limestone, shale and sand of the Jurassic age. Stratigraphically it is found that the formation on both abutments is Samana Suk Formation with Limestone and sand, shale beds. The diverse geotechnical studies were conceded out on the samples (disturbed and undisturbed) from the location, to determine the competency of the rocks and soil. From field tests and lab tests it is concluded that the Unconfined Compressive Strength, Point Load Strength Tests and Petrographic Analyses on samples shows high mechanical strength of rocks. Grain size outcome displays the existence of gravelly strata as major material encountered at minimal depth i.e. 1m to 5m on site. The bedrocks encountered at maximum depth >25m. The samples taken from the outcrop section and borehole that is the bedrock for dam foundation. Petrographic analyses shows a high percentage of ooids and pesoids (60% to 80%) and the process of micritization. The Atterberg's result displays the state of plasticity of the soils ranges from 5% to 10% (Plastic Index) and 25% to 30% (Liquid Limit) that is 'Slight- Low-Medium' which also corresponds to the Casagrande's arrangement which shows that most of the soils are Inorganic soils with Low plasticity. The permeability of soil samples that is ranging from 1.1178×10^{-4} cm/sec to 2.088×10^{-5} cm/sec which demarcates very low permeability with very slow class classifies the samples as impervious soils, which is an important parameter for the appropriateness of dam structure. The shear strength of the soils displays uniformity of the samples as 'very hard'. Test pits samples collected, in order to check the availability of construction materials nearby has shown the presence of A-4 material. As per AASHTO classification which can be used as construction material.

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CHAPTER-01

INTRODUCTION

1.1 Introduction

Dams are amongst the predominant and highly significant in engineering configurations construct for different purposes (Baxter, R. M. 1977). They are of varied proportions, forms and categories. In all circumstances, many necessary studies should be conceded out before determining the site, type and size of the dam. Amongst those studies is the geological studies which should be carried out to presume the geological situations in the most pertinent site, depth of the bases and their types, cut-off depth, type of the accessible construction supplies and type of the predictable geological risks. One of the most important parameter for dam sites are the geological investigations which should be carried out before selecting a dam location. Hence, there should be a complete investigation while constructing a new dam these investigations may include the abutments, foundation and reservoir's that must be explored and completely investigated to full support the geological aspect for respective dam site, without appropriate geological investigation the setting of a dam will cause severe hazards during construction and during working of the dam. For geological, geotechnical, hydrological and topographic motives, there are very little availabilities to construct dams at perfect location. It is well known that we have various types of engineering structures that are premeditated, assembled with lifelong probability (Xu Xiang-zhou et al., 2004). Generally speaking the foundation for the dam itself and the other structures to be build i.e. spillway, embankments and abutments are more enough durable and strong to support the stability of dam in future perspective or not, as we know once the dam is build up it will automatically occupy the whole area's geology. Infect the dam constructions serve marvelous purpose to the humanity, project and building of a dam is probable to create a rigid structure that will last for a very long period of time. Out of the various natural features that unswervingly effect the design of dams, one of the important than is geological, which not only control the character of the foundation but they also direct the materials accessibility for construction. However, for geologic, hydrologic and topographic reasons, there are limited numbers of ideal sites for dam's placement. Furthermore, petition for dam purpose of water source principally in areas with good potential is continuously increase. It is therefore very important to take-out satisfactory pre-construction investigations.

For support of dam site investigation, the information collected from the study of dam stability measures, dam envisioned for water supplies require a low restraint of leakage loses that is why stability of the dam matters for an ideal dam (Zheng et al., 2021). In addition to the design of damstructures must be enhanced to the current dam conditions to lessen the losses. Unable to do any of these may continuously result in unexpected seepage and/or total loss of the structure. Baz Ali dam is projected in the District Lower Kurram, Khyber Pakhtunkhwa, Pakistan. Geological and geotechnical investigations among other studies are mandatory for a safe hydraulic design and consequent execution of the project. Petrographic analyses were carried out on the area with corresponding Geological formation in order to support the feasibility and stability of the dam layout. The geotechnical investigations basically covered the drilling of boreholes and performance of in-situ tests in boreholes & assortment of soil samples from drilled boreholes, Excavation of test pits and assemblage of samples performance of laboratory testing on particular soil samples. Those soil samples were actually to provide the general permeability aspects and to provide the construction materials during the process of constructing the reservoir.

1.2 Location and accessibility of the Dam Site

The research area is situated in Lower Kurram. The area is about 31 km away from the Sadda town in Kurram (Organization Sub-Headquarter). Latitude of 33^0 38′ 20″ North and Longitude at 70^0 14′90″ East is the location of dam site. It is a dam for irrigation purpose that is a developing assignment in the water division which would deliver immediate alliance with the people of Kurram by posing occupation, recharged water table and enrichment sustenance crops. The proposed dam requires the persistent stream and overflow from numerous streams. The summit that have been design for the Baz Ali Dam bedstead with shoulder comprehending river and zoned embankment isabout 24.30 m, spillway core and material of impermeable clay that is actually showing positive signs for the dam.

The dam site frequently superimposes on sedimentary limestone basis having color from gray and lustrous gray. The quaternary deposits that is well known recent deposits of clay, shale, silt and loose unconsolidated fabrics with limestone on each site of Nainawarkhwar are unfossiliferous and to some extent as some fossils have been identified having an assorted formation. These deposits can be seen on both sides of the dam that is on the right abutment and left abutment respectively. In this order the dam abutments need to be raised on rocks having limestone which is abstemiously weathered.

Figure: 1.1 Accessibility and location of the studied area (Baz Ali Dam), (Google Earth).

1.3 Aim and objectives of the current research work

The aim of this research work is to present work out on the feasibility of Baz Ali Dam, Geologically and Geotechnically with respect to the stability of dam body. The main objectives of this research work are summarized below;

- It is to observe the lithological distribution of the rocks present in the area.
- To recognize and label the structural discontinuities, if there such as faults, folds and solution channels present in the rocks of the research area.
- To determine and evaluate the permeability of rocks masses that will be the foundation and structure for the reservoir.

- Petrographic analyses on the rock samples taken from boreholes and exposed rock units in order to check out the competency of rocks if they are more enough competent to support the dam body.
- On the basis of petrographic analyses it is to mark the discontinuities and evaluate the mechanical strength of rocks within.
- To examine the geotechnical properties of the rock and soil with respect to the dam foundation and the reservoir area.

1.4 Methodology

According to the scope of work Geological and Geotechnical investigations were performed to setup the overall research outcomes in a standard way. A detailed fieldwork was arranged to the selected area (District Lower Kurram, Khyber Pakhtunkhwa, Pakistan). The area was studied with Geological perspective and according to the study plan that is suitable for a reservoir build up. Starting from the top portion that is the upstream side the rocks were encountered. Likewise, the abutments were studied and wherever necessary samples were gathered for further investigations. Several structures were identified and noted with clean pictures. Structural discontinuities were noted accordingly. The dam body was fully analyzed. To evaluate the stability samples were gathered from respective borehole and sideways formations. To work on the permeability of dam body boreholes were drilled and different field tests were performed, these tests includes Constant head permeability tests, Standard penetration tests/Cone penetration test and packer's tests, Unconfined Compressive Strength and Point Load Strength test were achieved in the chosen boreholes and in agreement with appropriate A.S.T.M standards (Schmertmann et al., 1978). The exploratory borehole were drilled using Rotary and Heavy Percussion technique accordingly depending upon the subsurface features and physical appearance of the area. These Test pits were performed in order to provide construction materials availability in the vicinity of area. Soil samples were get together from selected depths from test pits using suitable sampling methods for the documentation and consequent laboratory testing. Designated soil samples were exposed to numerous laboratory tests for cataloguing and assessment of strength.

1.4.1 Field Instruments

The outcrop features of the dam site were examined using conventional methods of geological field work i.e. using Brunton compass to compute strike and dip data from the respective formations encountered, GPS that is to locate the area, quantifying tape in order to measure formation thickness or the structural discontinuities, geologic hammer and hand lens. Field photographs of indicative field topographies and field features were captured to provision the laboratory examination and understanding.

1.4.2 Sampling

It is very essential to have samples from the area where the field is accomplished, the samples that have been collected are actually the representative units of respective formation, samples from boreholes and samples from test pits. Initially the samples are designated from a greater mass or capacity to help as an example of that superior body or to reproduce some detailed feature or dissimilarity insideit. The simple stations for specimen is that, one can take a sample home, but not an outcrop (Barnes, 1981).

1.4.3 Core Drilling

Core drilling in the rocks was completed with straight rotary rigs. Casing was used in overburdento prevent the borehole caving. Clean water was used as drilling fluid. Core barrels of double tube, steelbits were used to produce “NX” size (Smith et al., 1959). Samples were taken and those samples obtained from the boreholes were preserved in standard size core boxes.

1.4.4 Standard Penetration Test

Standard Penetration Test is actually an in-situ examination that is approaching beneath the group of penetrometer tests which is well known to us. (Roger et al., 2006). The Standard Penetration Tests are carried out in the boreholes. For the penetration experienced the test were used to quantify the resistance of the soil layers. A standard split spoon sampler is essentially used for borehole preparation and for complete processes performing standard penetration tests. After completion of boring to the wanted depth, the drilling instrument is detached and the sampler is located inside the borehole. Through the droplet of hammer of 63.5 kg mass dropping from a height of 750 mm at the rate of 30 setbacks per minute, the sampler is obsessed into the soil. The number of blows of hammer are compulsory to drive adepth of 150 mm is calculated, further it is driven off to 150 mm and the blows were calculated. Likewise, the sampler is again extra driven by 150 mm and the number of blows were verified. The recorded number of blows of the first 150 mm not to taken into contemplation. Thus the number of blows logged for last two 150 mm intermissions are sum up to give the Standard Penetration number.

1.4.5 Test Pits and Field Density Tests

For geotechnical investigations and construction materials availability it is necessary that in the vicinity of area test pits and field density have to be performed (Van Rooy et al., 2001). The size of the pit will be such that a person can easily enter the pit, having a visual inspection and can carry out FDT (Field Density Test) (Mujtaba et al., 2020). Test pits were digged manually. After all soil samples were collected from the pits for detailed analysis.

1.4.6 Field Permeability Tests and Water Pressure Tests

Field Permeability Test i.e. performed in order to have the results from the subsurface rocks and the strength of their permeability. Field Permeability Test are performed by using constant head permeability test. Water pressure tests provides a degree of the receipt of water by rock in-situ below pressure (Yihdego et al., 2017). This inspection was formerly presented by Lugeon to deliver satisfactorytypical values for the permeability of dam foundation. Principally, it contains the amount of the capacityof water that can discharge from an uncased segment of borehole in assumed period under agreed pressure, however the single packer method is suitable for soft rocks where the problem of caving is common and for sound rock, double packer method is more suitable. The main shortcoming of dual packer process is that leakage through inferior packer may go unobserved which typically over guesses the permeability. The test is used to admittance the quantity of filling that rock will receive, to check the efficiency of grouting, to get an amount of the rupturing of rock or to give an estimated worth of the permeability of the rock mass.

1.4.7 Groundwater Observations

Groundwater measurement method is dependent upon the scope of work. Groundwater observations are mainly done by straight Rotary and Percussion drilling that were used accordinglydepending upon the behavior of soil/rocks.

1.5 Laboratory Analyses

Samples taken from the studied area a laboratory tests were performed in order to evaluate them and brought out the information that are in need. Samples obtained from field outcrop and from boreholes respectively have to be analyzed in laboratory.

1.5.1 Thin Sections Preparation

Thin sections for petrographic observation were prepared in Earth Sciences Department of Quaid-i-Azam, University, Islamabad. Thin section are actually used for petrographic analyses of samples. After preparation of thin sections samples are studied under microscope to find out textural and mineralogical properties.

1.5.2 Petrographic Analyses

Generally speaking petrographic analyses are performed in order to study the sample's texture, mineralogy, microfacies and for fossils analyses. Petrographic analyses for this research work were carried out in order to find out the micro fractures, assemblages of minute structures and the overall texture and mineralogical composition of rocks whatever it may be in order to exaggerate the stability of dam body.

1.5.3 Unconfined Compression Tests

For laboratory assessments Unconfined Compression test used to develop the Unconfined Compressive Strength (U.C.S) of a rock sample (Wyering et. al., 2014). Unconfined Compressive Strength (U.C.S) postures for the extreme axial compressive stresses that a sample can tolerate under zero restraining stress, the stress is functional laterally be longitudinal axis due to the point, It is also recognized as Uniaxial Compression Test. While performing this test, separately from the axial load and lateral alteration are usually measured to process the elastic modulus and Poisson's ratio of samples.

1.5.4 Liquid Limit and Plastic Limit

These were essentially established on the basis of Atterberg's limits which comprise of the subsequent key values of moistness content:

(L. L) is the moisture content at which is elongated or drifts alike liquid due to fined grained soil. The (P. L) is the moisture content at which can no longer be remolded without outrageous of fine- grained soil. (Victor N. Kaliakin, in Soil Mechanics, 2017).

1.5.5 Point Load Strength Tests

The P.L.S. tests actually defines the complete rock strength catalogues from the outcrop or drill core samples in geotechnical witnesses (Rusnak et. al., 2000). The point load strength tests device and technique permits reasonable testing of core or lump rock samples in either a field or laboratory background.

1.5.6 Particle Size Distribution

We can call it sieve analysis basically it is used for the size distribution of a solid particle by describing the quantity of fine particles retained on a sequence of sieves with respective various sized openings. A sample is advance to the top of a shells of sieves that is organized in decreasing order from top to bottom, as the sieves are shaken, the sample is isolated onto the different sized sieves. The size distribution and mean width of the sample and weight of samples engaged accordingly which is govern by sieve.

1.5.7 Maximum Dry Density and Optimum Moisture Content

Dry density and optimum moisture content of the soil is actually used for the amount of compaction level (Spagnoli et. al., 2020). Standard proctor compaction test and Modified proctor compaction tests are mainly use. These tests helps to control the ideal moisture content which are compulsory for soil to reach maximum compaction i.e. conclusively maximum dry density used for accomplishment of construction.

CHAPTER-02

TECTONIC SETTING AND STRATIGRAPHY

2.1 Tectonic Setting of Pakistan

It is prior to mention that the supercontinent that is Pangea divided into two supercontinents about 200 million years ago that is Laurasia and Gondwana to the north and south respectively, which is known to be separated by the early Paleozoic Tethys Ocean (Du Toit, 1937; Boulin, 1981). The Indo-Pakistan sub-continent is actually bounded by Indian Ocean and by Himalayas from south and north respectively. It is known that Continental drifting, sea-floor spreading, and collision of tectonics are most important natural factors that are known to be the reason that the formation of Himalayas and the collision of Indian with Eurasian plate (Jhonson et al., 1976; Coward et. al., 1986). After these activities happened early we say that the Eurasian Plate primarily composed of Europe and Asia, while the Gondwana Plate were mostly comprised of Antarctica, Australia, Arabia and Africa. It is an important factor to mention that amongst these two major continents there were presenting a small continental fragments, which were composed of Precambrian and Paleozoic ages, dividing the “Tethys” into two new oceans, that is in the north “Paleo-Tethys” and tin he south known” Neo-Tethys” (Smith et al., 1981). The collision between Indian Plate and K.I.A occurred at Eocene time that is when the two “Neo-Tethys” were closed in South (Tahirkheli et al., 1979). Indian Plate subducting beneath the Eurasian Plate in the direction of North direction is known to be still in progress (Seeber et al., 1981) and this subduction is resulting to form major tectonic fabrics of Pakistan that primarily consist of the MKT, MMT, MBT and SRT.

2.2 Regional Tectonic Setting

With respect to Geological point of view the area is composed of Precambrian age crystalline rocks of the Indian and Kabul blocks, Sedimentary rocks of Permian to quaternary age, Paleocene Kabul - Altimur and Zhob-Waziristan - Khost ophiolites complexes in bulk from Late-Cretaceous age (Beck et al., 1996). Himalayan collision along the Chaman Transform Faulted boundary and the extrusion of the Kabul block was the result at that time, it is worthy to mention that Katawaz Basin is composed of clastic strata of Early-Indus fan (Badshah et al., 2000). Southward thrusting of the Spinghar Indian crystalline basement rocks over the Pliocene Murree Formation are the current compressional debates. Six units in the Spinghar and South Waziristan were demonstrated on the basis of Geological analyses novel designations and type sections. The results are the Spinghar Quartzite, Daradar Dolomite, Sikaram series, Makral Limestone, Wana Schist and Kaniguram slates with referenceto (Malkani et al., 2017).

The Kurram is observed as by highly mixed, multifaceted geology. This is why the Kurram Agency has been separated into three tectonic blocks (Brookfield et al., 1998). Spin Ghar Block, Samana Block and the Kurram-Waziristan Block in the north, middle and south directions respectively,The Murree Formation that is known to be initially a tectonic contact is well exposed in the Makai Thrust with a narrow belt ungluing the Samana Block from Spin Ghar Block respectively, however on south of Samana thrust the Kurram-Waziristan Block is dominantly visible. On south of Samana Thrust the K-W Block is a thrust sheet, which is mostly well and covering the whole western edges of the Samana Range by succeeding deformation in the Samana Range into E-W trending doubly plunging anticlinal fold structures and associated thrusts lifted. The tattered superimposing Kurram- Waziristan thrust sheet with conservation of the leftovers only as klippe with respect to the western Samana Ranges in the Kurram Agency that is mainly self-possessed of Mesozoic-Early Tertiary lithologies alike to those exposed in the Hill Ranges to the east (Kotal-Kalachitta-Margalla). In the Samana Ranges the native remnants of the Kurram-Waziristan thrust sheets come across. Basically the Spin Ghar Block consists of five major lithological units, which are directing from north to south are: Spin Ghar Crystalline Complex, Daradar Dolomite, Spin Ghar Quartzite, Sikaram Series and Makai Limestone respectively (Malkani et al., 2016).

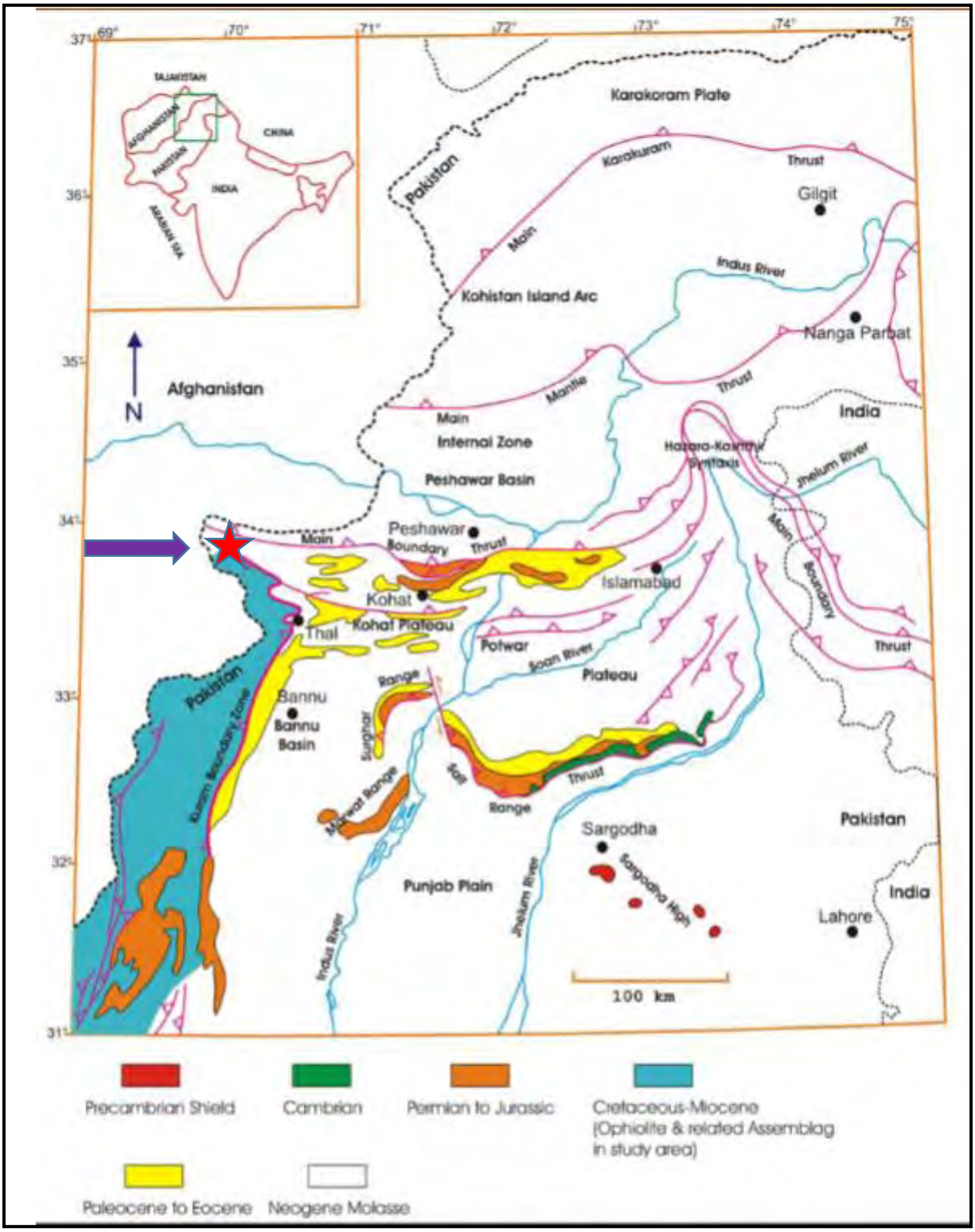


Figure: 2.1 Map of the western Himalayas, North Pakistan, showing the location of Main Boundary (MBT), Kohat-Potwar Plateau and Kurram Boundary Zone (KBZ) (Khan et al., 2003).

2.3 General Geology and Stratigraphic Setting

2.3.1 Samana Block

The Samana Block is known to be comprised of the lithostratigraphic units that is pointing to the south of Spin Ghar Crystalline Complex and on the footwall of Makai Thrust. It is composed of two groups of lithological units, which is said to the part of two thrust sheets, that is the below thrust sheet and the above thrust sheet. The lower thrust sheet is actually comprising the India-Plate shelf sequence that is known to be from Triassic to Miocene in the outer lesser Himalayas commonly exposed in the Samana, Kalachitta and Margalla ranges. The rocks from Cretaceous age are the basal for the upper thrust sheet and is well-defined by the Kurram Block (Beck et al., 1996).

2.3.2 Mesozoic Rocks

The Mesozoic era rocks are ranging from Triassic to Cretaceous period. The Samana Suk Formation of Jurassic age, Chichali and Lumshiwal and Darsamand Limestone of Cretaceous age comprised of Mesozoic era rocks of the Indian-Plate Shelf Sequence. These are actually the rocks exposed at the vicinity of dam site. It is worth mention that in the provincial setting of Baz Ali Dam site. While discussing the Geological setting of the studied area it falls in the Parachinar–Kalachitta fold belts and so it is recognized to be situated north and North-West of the Kohat Potwar Plateau with reference to (Ahmad et al., 2006). The interaction amongst ranges of the plateau has an important thrust fault known as Parachinar Kurram thrust in Parachinar Kurram belt and Parachinar Murree area. The studied area has its own lithological and structural distinctiveness it is due to the reason of neighboring location with Pukhtia Province of Afghanistan. It is why it can greatly be distinguished from the Kohat Potwar areas,but it can be categorized with the exposure of Jurassic age rocks and Cretaceous age that is unlike the Kohat Potwar areas.

2.4 General Stratigraphy

Generally this area is occupied alongside the Western border of Pakistan and it is known that the area is covered with sedimentary and carbonate deposits ranging from Jurassic to Pliocene. As the area is underlain by sedimentary and carbonate rocks, geographically in the northwestern corner that is near Afghan border basalt sills intruded rocks which are expecting to be from Jurassic to Cretaceous. It is observed during the field that the transported rocks found there at the studied area were which actually derived from the extreme West of Pak-Afghan border that were the metamorphosed sedimentary rocks. Broadly in the northwestern part of Parachinar quadrangle main rocks of mélange zone of Cretaceous age thrusting above the east ward Jurassic folded rocks, Cretaceous age and the Paleocene strata. The stratigraphic arrangements are written below.

2.4.1 Jurassic Sequence

Jurassic Sequence mostly comprised of gray to dark grayish, thin bedded to thick limestones covering intercalated sandstone, shale & marls. Rocks are partially metamorphosed. Igneous rocks are exposed in the west of Parachinar (Pivnik et al., 1996). The Shinawari Formation, Samana Suk Formation are present in the study area. They are written below in detail.

2.4.1.1 Shinawari Formation

The name was firstly formulated by Fatmi and Khan (1966) after a village located in the western part of Samana ranges considered type locality for it. It comprised of thin bedded and massive bedded limestone of gray colour. Limestone is interbedded with subordinate shale, sandstone and mudstone. The colour of shale gray to yellowish, calcareous and splintery. The most prominent lithology is massive dark-gray oolitic limestone. The limestone has karstified texture and it has rotten smell on fresh surface. It ages from lower to Middle Jurassic age Fatmi and Cheema (1972).

2.4.1.2 Samana Suk Formation

Initially it was known as Samana Suk Limestone in the Kohat District and the Orakzai Agency Davies (1930) formalized later by Shah (1977) as Samana Suk Formation. It comprised of light gray to gray, dark-gray, thin bedded to thick bedded and massive limestone. Minor amount of reddish gray sandstone and shale. Limestone is partially recrystallized and marblised with stylolite have been developed. The Limestone is sandy and shows lamination at some intervals.

2.4.2 Cretaceous Sequence

The cretaceous age rocks exposed in the studied area were generally specified by glauconitic sandstone, shale, thin bedded to thick bedded limestone and interclations of calcareous shale with marls. The Chichali Formation, Lumshiwal Formation and Darsamand Limestone are the formation exposed inthe area.

2.4.2.1 Chichali Formation

It was initially introduced by Danilchik (1961) and Shah (1967) for the rocks in Trans Indus Ranges and the Salt Range by Spath (1938) and Gee (1945). It is dark green, glauconitic sandstone with gray silty glauconitic shale and it is massive and contains sandy shale at places and seems to be hard, light gray to white quartzose touch.

2.4.2.2 Lumshiwal Formation

It was named Lumshiwal Sandstone by Gee (1945) later it has been formalized as Lumshiwal Formation by Shah (1977). It is 1 km north of Lumshiwal Nala by Danilchik and Shah (1967). It is mostly gray to light gray, thick bedded to massive bedded and the current bedding contains feldspathetic sandstone and silty/sandy glauconitic shale.

2.4.2.3 Darsamand Limestone

Late-Cretaceous age rocks exposed were named Darsamand Limestone by Fatmi (1973), basically found on the synclines and on the flanks of anticlines in the northeastern part of Parachinar, the formation is divided into two members the lower and upper member. In detail descriptions lower member is light gray-olive gray, thin bedded to thick bedded, and having dense to soft nodular beds of marls and shale, the actual thickness of the formation is about 200 ft at this below part. The above part is thick bedded to massive bedded, gray and dense, thin beds of calcareous shale are recognized, the actual thickness of the formation at this part is about 180 ft. Upper contact is gradational with Lumshiwal Formation and Lower contact is unconformable with Hangu Formation.

2.4.3 Tertiary Sequence

Tertiary age rocks are mainly composed of the Paleocene, Eocene, Oligocene and Miocene. In the study area Paleocene age rocks were exposed that were studied that is the Hangu Formation.

2.4.3.1 Hangu Formation

Initially it was named as Hangu Shale or Hangu Sandstone by Davies (1930) after then formalized as Hangu Formation by Shah (1977). It consists of sandstone, gray colour shale with minor amounts of mudstone and claystone, carbonaceous shale in the upper part and some nodular limestone, coal beds and minor limestone in the lower part was noted at that time. It is of Early Paleocene age by Davies et al., (1937) and Haque (1956).

2.4.4 Mélange Zone

The western part of Parachinar quadrangle and Afghanistan border are essentially covered by Kurram Formation that is situated in mélange zone. Lithologically it is comprised of red, brownish red, gray shale, mudstone and siltstone. Thin bedded to thick bedded limestone of grayish color and argillaceous at places. Cherty beds were also found in some places.

2.5 Surface Lithology of Reservoir

During the field looking at the site the continuation of sedimentary rocks and soil units have been investigated, it is also noted that there was no influence of metamorphic or igneous activity. It is well- known that the huge transported boulders and cobbles of igneous and metamorphic sources are found in the stream bed. These rocks in the research work area are actually describing the importance of these rocks wherever in the catchment range of Nilawahan Khwar and perhaps attached branches that have been transported by flood and glacier. Moreover the bed rock lithology of Baz Ali Dam comprises mostly of sandstone, shale and thin to thick bedded limestone. Looking closely to these rocks they are moderately to high weathered mostly covered at places the rocks exposures which give rise to stunted vegetation in the surrounding area.

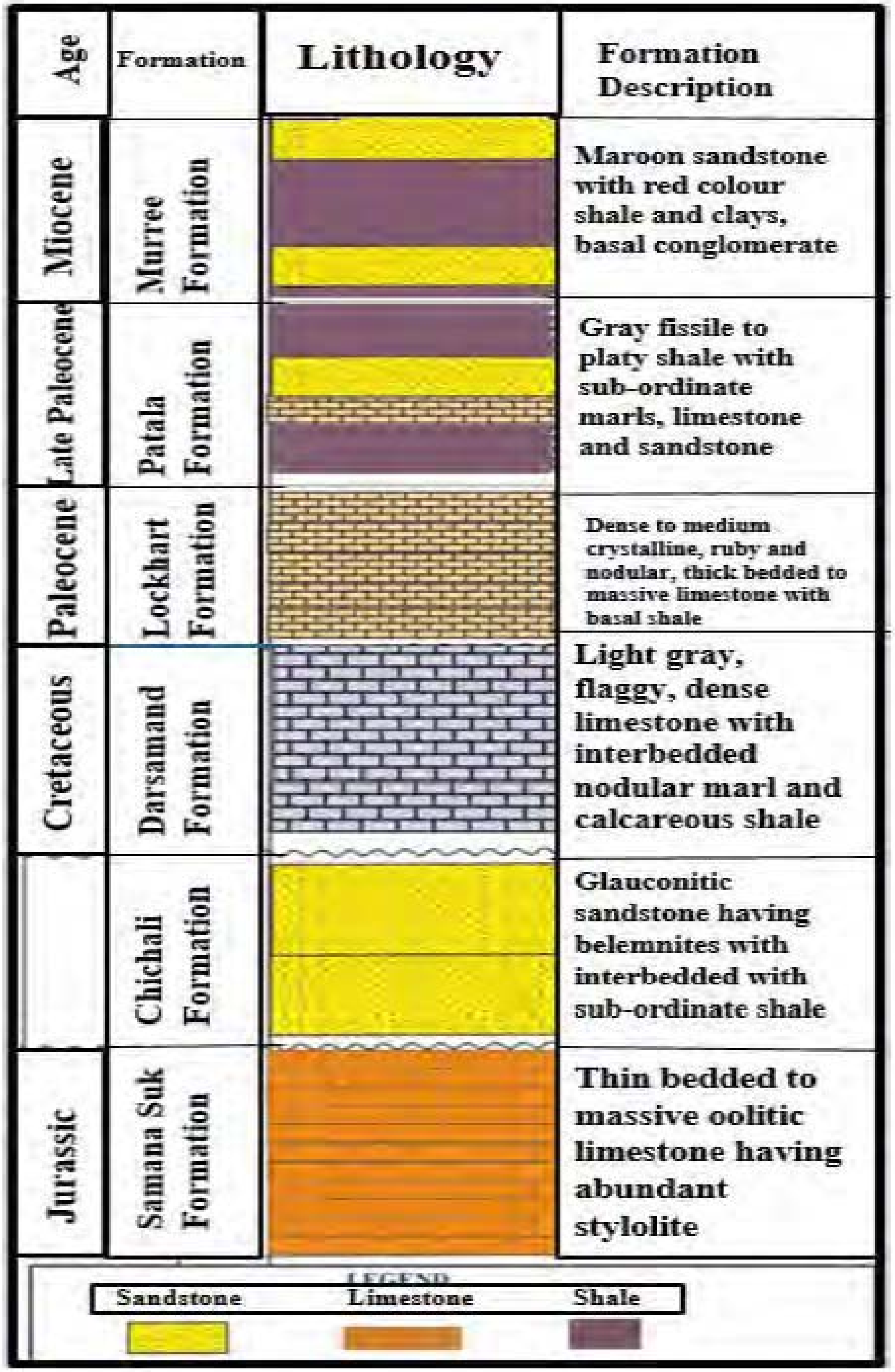


Figure: 2.2 General stratigraphy column of Kurram area (Khan et al, 2003).

CHAPTER-3

OUTCROP STUDIES

3.1 Introduction

A detailed fieldwork was conducted to the planned area i.e. Lower Kurram, Baz Ali dam. The main theme of visiting the site was to collect samples, to observe prominent features present in the area and to investigate the area. The overall geological and geotechnical investigations are discussed below;

3.2 Walai Chena, Sadda, Lower Kurram

Broadly speaking in the vicinity of proposed site there were found numbers of complex structures that were S-shaped folds, Z-shaped folds, apparent discontinuities, stylolite, joints, fractures, bedding discontinuities and calcite veins that is linking to the Jurassic age formation namely Shinawari Formation and Samana Suk Formation below in figures.

3.3 Field Features Observed

Field features were observed accordingly from the proposed site. The overall area of the proposed dam site that starting from below part i.e. downstream to upstream and along the both sides of dam i.e. right abutment and left abutment with spillway. The abutment's rocks were composed of Samana Suk Formation majorly comprised of limestone with sand and interclations of shale and marls of Jurassic age. The overall formation is about 20m in thickness at type locality in figure 3.1 (A). The below part is composed limestone, the colour is gray on fresh surface and it is green to yellowish green on weathered surface, fossiliferous in figure 3.2 (E). Same lithology continue, the beds are decreasing in size up to some extent as we move to the top of formation. The colour on weathered surface is dark gray and on fresh surface it is green to yellowish. From 5m to 10m the formation showing interclations of shale beds seems to be muddy in colour and marls with same distinctive colour, the colour of limestone at this bed seems to be lustrous shiny with fossil assemblage is less then below part in figure 3.1 (C). From 10m to 15m same lithology but more exposed to weathering that's why honey comb weathering take place with thinly laminated shale showed in figures 3.3 (J). From 15m to 20m the limestone beds are massive bedded and highly weathered and covered by irony stained. Overall the formation having some structural features that are joints and fractures in figure 3.1 (D).

Deep down on the right abutment, structures were identified that were highly complex showing structural features of S-Shaped fold and Z-shaped fold. Field photographs were taken from that part in figure 3.2 (F, H). They were comprised of clay, shale beds that seems to be slightly metamorphosed, phyllites and some beds of limestone in figure 3.2 (G).

At the top of reservoir that is on extreme left abutment some boulders of dolomite were identified that were moderately to highly weathered, medium bedded to massive bedded, highly fractured and jointed with veins of calcite. Butcher chop weathering was the major effect on these rocks in figures 3.3 (I,J,K).

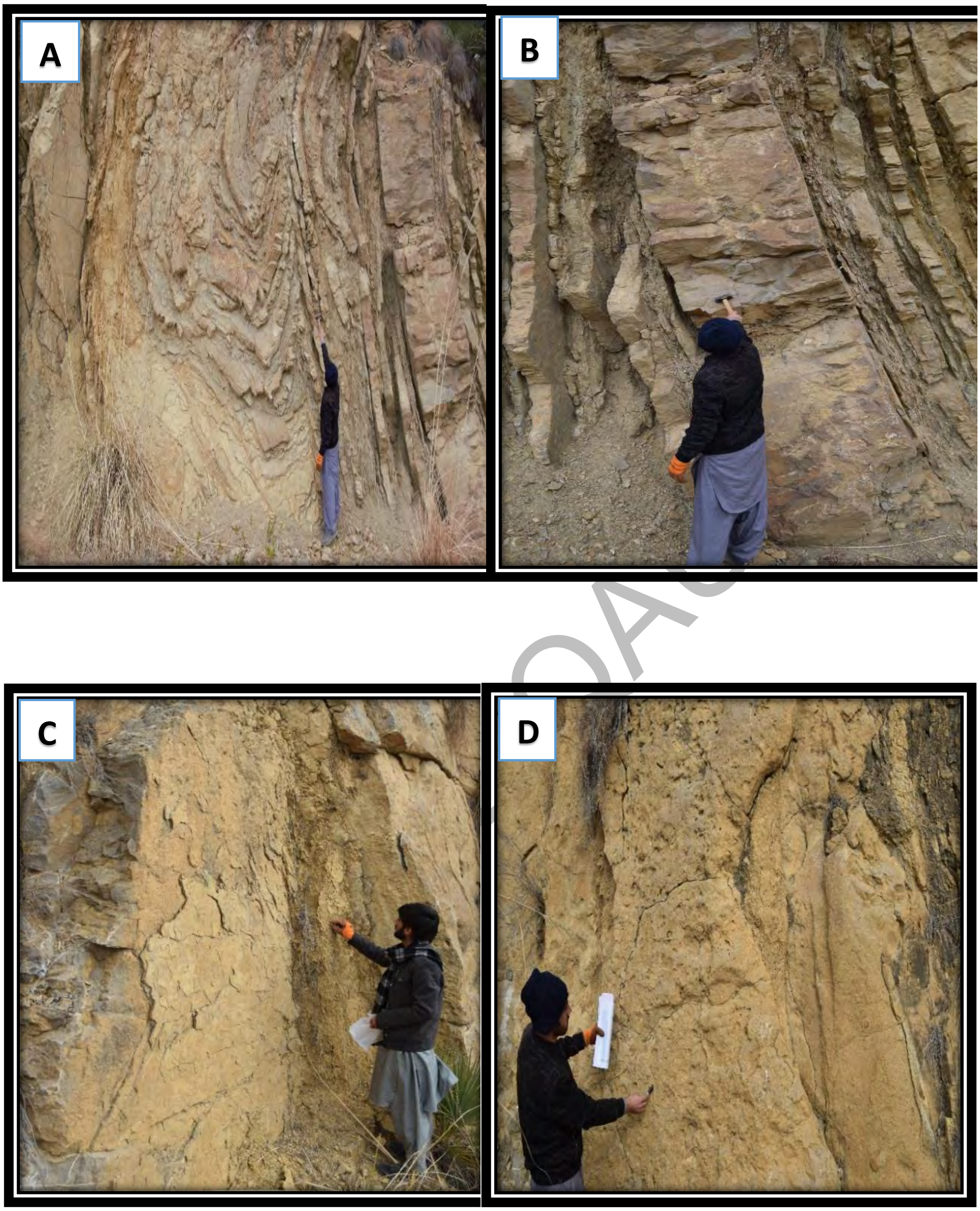
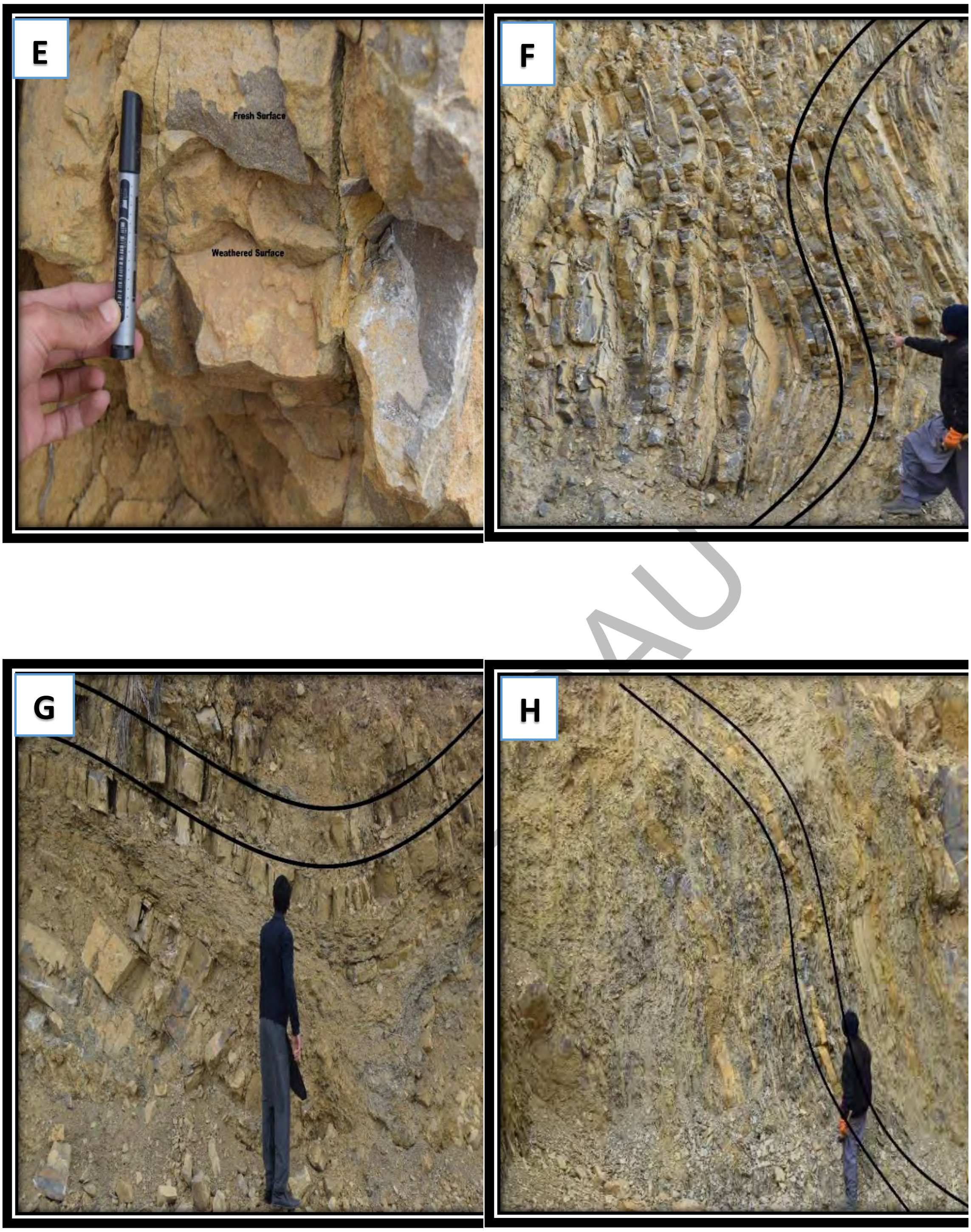


Figure: 3.1 Field features observed during the field. (A) Photograph taken from the outcrop section of Samana Suk Formation at Lower Kurram on left abutment of dam. (B) Photo showing the thick massive beds limestone of formation. (C) In the photo the shale and sand beds of formation are shown. (D) Photo showing the joints and weathered surface within the outcrop.


E
Fresh Surface
Weathered Surface
F
G
H

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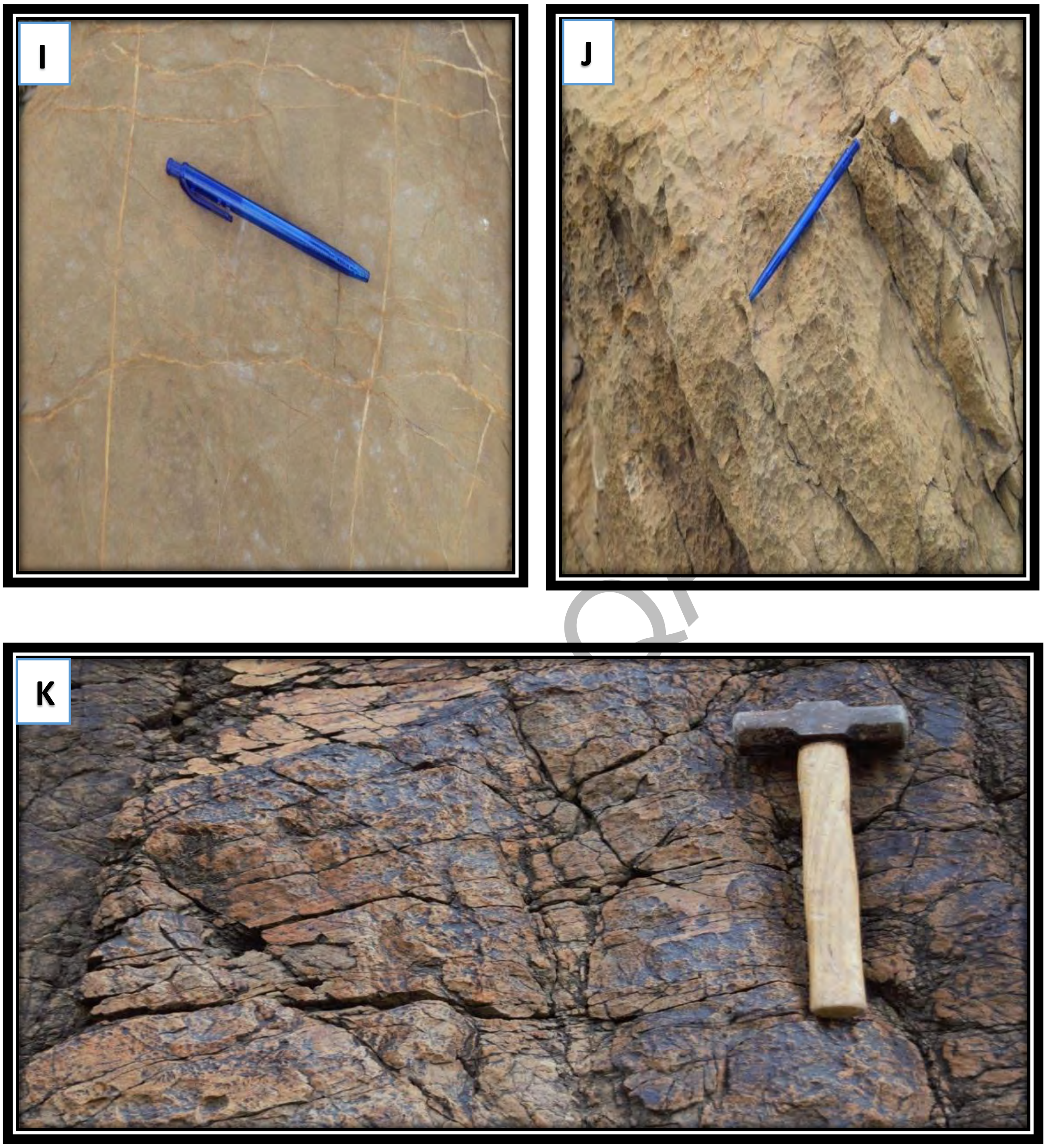


Figure: 3.2:- Photographs taken from the outcrop section of Samana Suk Formation at Lower Kurram. (E) Photo descripting the formation's weathered surface that is yellowish in colour and the fresh surface that is gray in colour. (F) Deep down on the right abutment S-Shaped folding features. (G) Photo showing the folding features within the abutment of dam site. (H) Z-Shaped folding phenomena.

3.4 Field Tests

A one month detailed geological field was carried out to the particular study area (i.e. Lower Kurram) separately in order to perform field tests. The field inquiries included the subsequent doings;

- Boring of investigative boreholes
- In-situ analysis in boreholes
- Collection of earthy samples from boreholes
- Excavation of Test Pits
- In situ Testing in Test Pits
- Collection of soil samples from Test Pits
- Constant head permeability test
- Water pressure test

3.4.1 Exploratory Boreholes

The purpose of exploratory and boreholes is to elicits and extract core of the subsurface rocks or soil to be examined practically as well as in the laboratory by National Research Council (1994). This is achieved by means of rotary core drilling and heavy percussion machinery accordingly to achieve the best possible results. The location of boreholes are given below in figure 3.4.

3.4.2 Core Drilling

Core drilling in rock was completed with straight rotary rigs. Casing was used in overburden to prevent the borehole caving. Clean water was used as drilling fluid. Core barrels of double tube, steel bits would be used to produce “NX” size (Smith et al., 1959). Core drilling started with 1.0-1.5 m run. After the completion of each run the fundamental container were detached from the borehole and the rocks were extruded from the core barrel. The rock samples were marked with the project name, borehole number, sample depth, date of sampling. The samples obtained from the boreholes were preserved in standard size core boxes in figures.

3.4.3 Standard Penetration Tests

It is well-known that SPT is actually a motionless examination that is approaching beneath the other tests i.e. penetrometer tests (Roger et al., 2006). These tests are carried out in the subsurface (boreholes). They were used to quantify the soil layers resistance shown to the infiltration experienced on it. A penetration observed relationship is derivative amongst the soil characterization and the resistance shown to penetration. The tests were tremendously valuable for defining the comparative density and the shearing resistance angle of soils in which cohesion is negligible. So it can be alternatively used to govern the UCS (unconfined compressive strength) of soils with cohesion properties.

3.4.4 Groundwater Observations

Groundwater measurement method is dependent upon the scope of work. Groundwater observations are mainly done by straight Rotary and Percussion drilling that were used accordingly depending upon the behavior of soil/rocks.

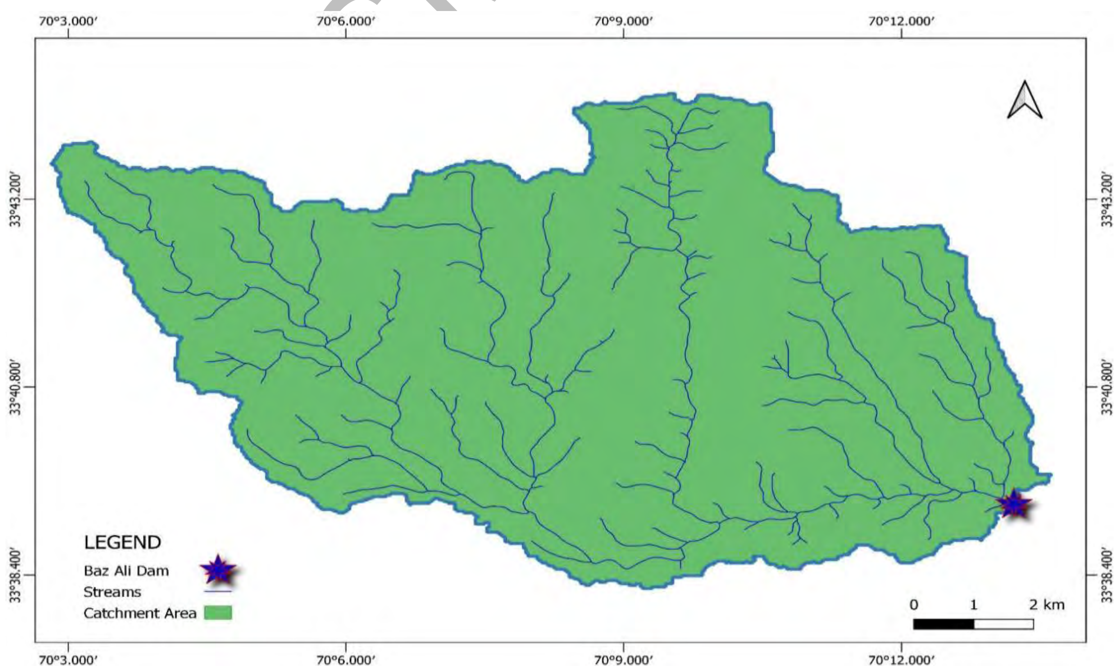


Figure: 3.4 Catchment area of Dam from nearby distributaries channels.

3.4.5 Test Pits and Field Density Test

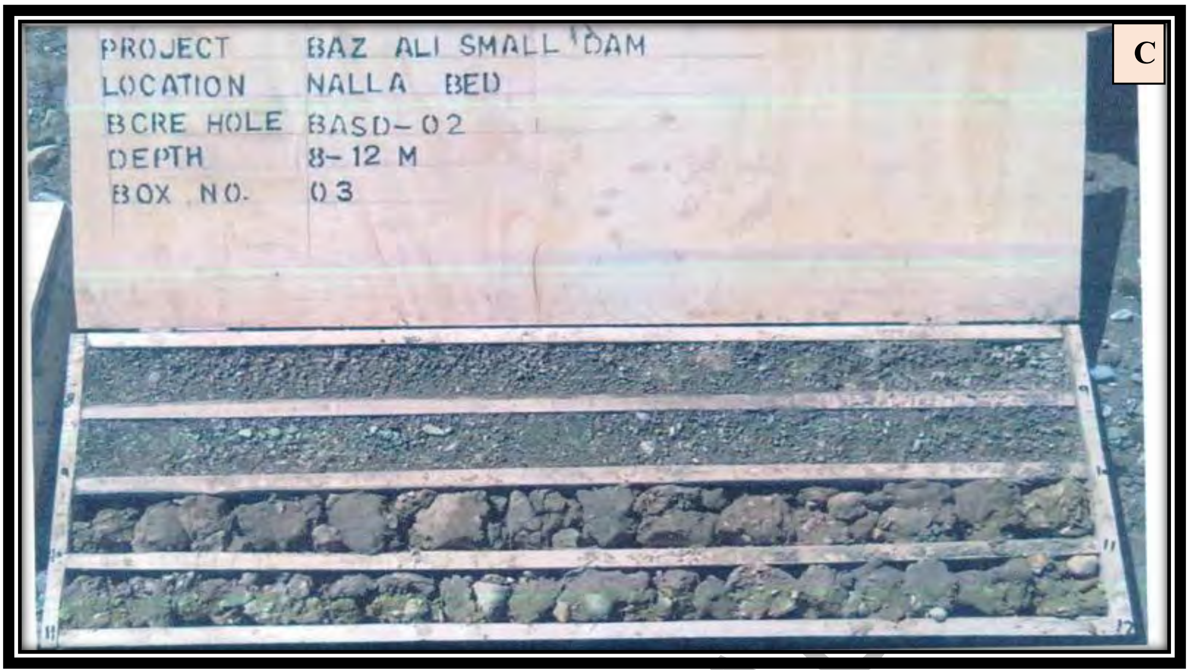
For geotechnical investigations it is necessary that in the vicinity of area test pits and field density have to be performed (Van Rooy et al., 2001). The size of the pit will be such that a person can easily enter the pit and having a visual inspection and can carry out FDT (Field Density Test) (Mujtaba et al., 2020). After all soil samples were collected from the pit for detailed analysis.

3.4.6 Field Permeability and Lugeon/Water Pressure Tests

Field Permeability Test i.e. performed in order to have the results from the subsurface rocks and the strength of their permeability. Field Permeability Test were performed by using constant head permeability test.

Figure: 3.5:- Figure showing the specific points where boreholes were drilled in the dam site and the arrows pointing towards the outcrop section from where the specific samples were gathered accordingly to the need of work.

Figure: 3.6:- Photographs showing the core samples from the boreholes. (A) Lithologically it is clay and gravel. (B) Lithologically it is more abundant in gravel with little sand.



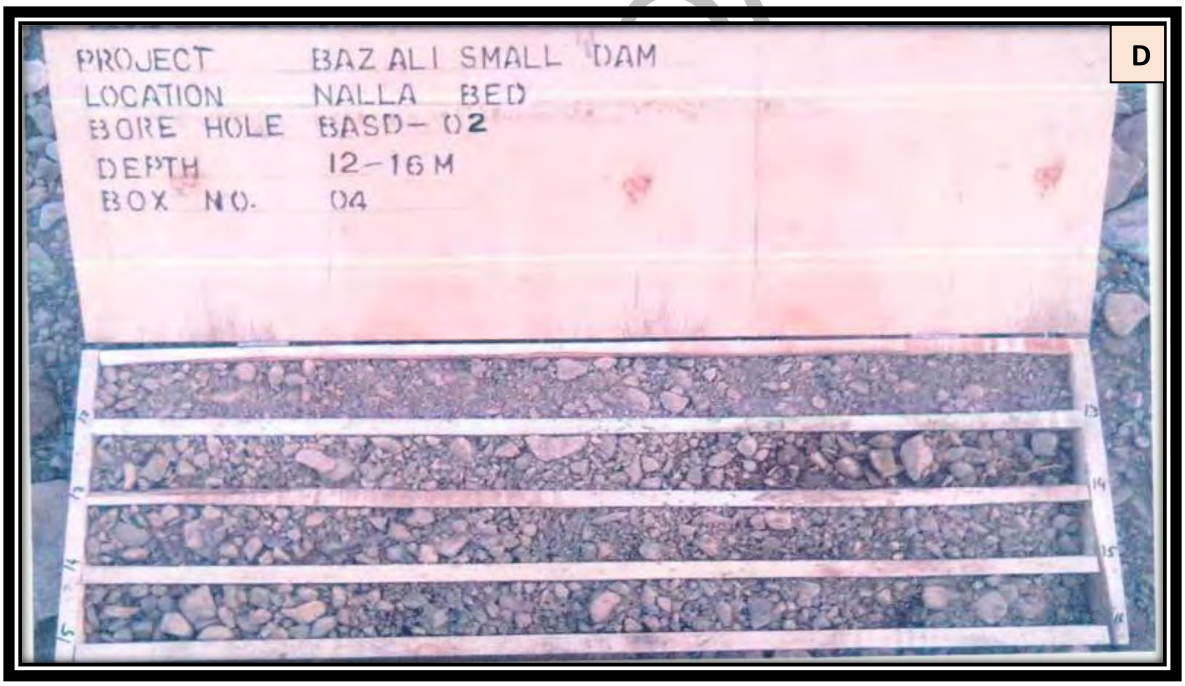


Figure: 3.7:- Photographs showing the core samples from the boreholes. (C) Lithologically it is clay, gravel and cobbles. (D) Lithologically it is more abundant in gravel with sand.

Figure: 3.8:- Photographs showing the core samples from the boreholes. (E) Lithologically it is gravel with greater amount of limestone. (F) Lithologically it is gravel, limestone with little amount of sand.

Figure: 3.9:- Photographs showing the core samples from the boreholes. (G) Lithologically the core box is filled with greater amount of limestone. (H) Lithologically it is gravel, sand and limestone.

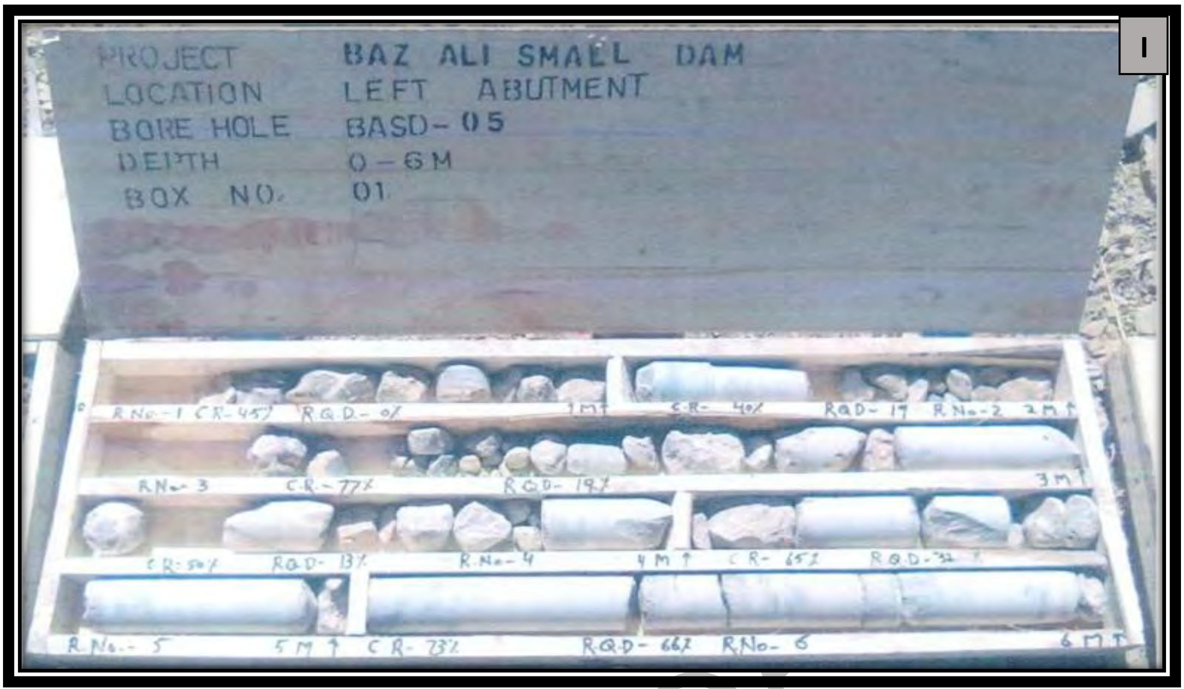


Figure: 3.10:- Photographs showing the core samples from the boreholes. (I) lithologically it is limestone. (J) Lithologically it is gravel, sand and limestone.

Figure: 3.11:- Photographs showing test pits progress. (K) Measuring the specific advised size of a standard test pit at the site. (L) The process of performing F.D.T in the field.

CHAPTER-4

RESULTS AND DISCUSSIONS

4.1 Petrographic Analyses

Thin sections were prepared and analyzed under petrographic microscope that is also known as polarizing microscope. Polarizing microscope is best described as a compound, transmitted-light microscope to which components have been added to enable the determination of the optical properties of translucent substances in detail. In this research work different laboratory test were performed on selected samples from the outcrop section and boreholes in the area in order to evaluate the strength of rock masses and their bearing capacity to support the foundation and withstanding of dam structure, these tests include UCS (Unconfined Compression Strength Test) and Point Load Strength Tests. Petrographic analyses were carried out on samples from respective area that were from outcrop and boreholes where necessary. Petrography of samples were carried out in this aspect to look for textural parameters and correlate with mechanical strength of rocks. For this a systematic approach was used in the field to collect the samples from location where there is the best possible possibility to encounter the strength of rocks for the competency that they would be able to support the reservoir in the future. It is well known that that Unconfined Compressive Strength, Point Load Strength and other mechanical properties showed medium to inverse relationship with sparite and allochems, whereas micrite and limestone and dolomite shows a positive correlation with U.C.S values. In order to get sufficient correlations for cumulative micrite, limestone, sparite allochems percentages and strength parameters were established in this research work.

A total of 12 samples were collected from specific locations that is 6 samples from outcrop sections and 6 from the boreholes which were at a depth of about 30m having that specified rock units of Samana Suk Formation. Petrographic interpretation of Samana Suk Formation shows ooids, pesoids, blocky calcite, micritization, muddy groundmass, grainstone, wackstone, packstone and sparite with different percentages. All these founding were actually the results of different stages of diagenetic events as mesogenesis and then burial occurs at the vadose/shallow zone of deposition the rocks filled by two methods (a) filled with veins (b) replaced by bioclasts. While this happening we assume that the replacement of fossils by bioclasts (Aragonite) is actually showing a plus point for the strong mechanical strength of rocks in the area.

Burial of rocks occurs as showing the mechanical compaction. As burial occurs the rocks in subsurface become slightly fractured and with this fluid released as a result stylolite produced, the concentration of water in this way become very low and the grain to grain compaction simultaneously the place which is a plus point for the mechanical compaction of rocks here, in this way when water released within the formation, precipitation occurs that alternately filled the veins and stylolite within it, filling of fractures is actually providing strong competency for rocks.

As ooids deformation take place within the formation it is not suitable for the rocks competency but the lagoonal ooids of Samana Suk Formation of marginal marine environment produced dense concentric ooids which are having multiple layers of enveloping shells that enveloping the nuclei of former pre-existing pellets. In this way the concentric ooids deformed at grain contact and go to less brittle deformation and thus in burial depth it retains the mechanical strength of rocks.

The petrographic analyses shows that the groundmass is matrix (muddy or lime mudstone) which is fine grained and we know that; strain is directly proportional to grain size by following this mostly of deep marine setting, which seems to be less in volume in Samana Suk Formation as it is in inner ramp setting. The later stages of telogentic calcite veins related to detailed study of thin sections shows unwanted marks for less potential of rock and can decrease the stability dam infrastructure but here it is said that the volume of these fracture is very much less in average and it would not affect the stability of foundation as these are filled out by pre-formed fracture which known to be formed during uplift stages. Ooids are more enough hard and supportive that it could withstand the uplift thus it is obvious that Samana Suk Formation is having high strength. The bioturbation vanish the original accumulation of ooids and pesoids but it is later filled with micrite thus there is no open voids, the micritization supports the mechanical strength of rocks.

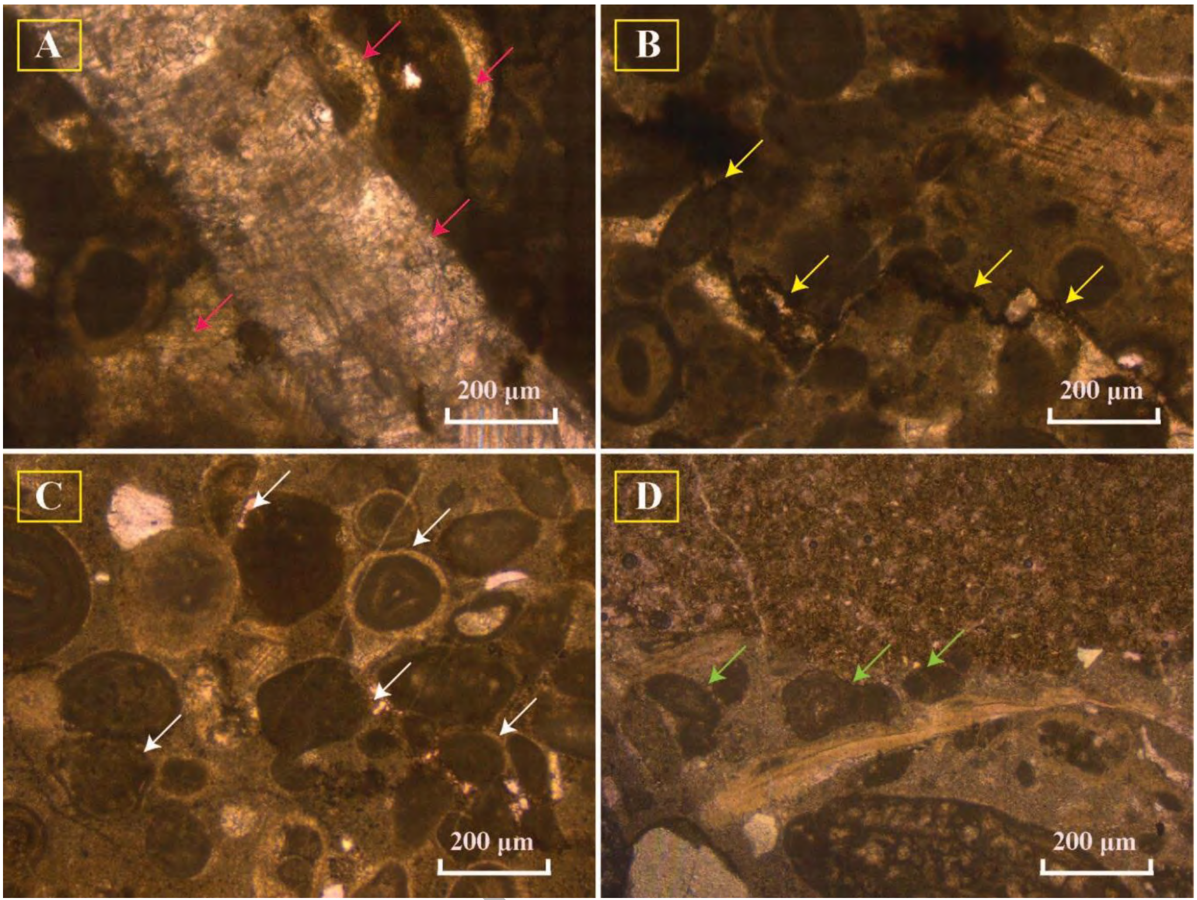


Figure: 4.1:- Photomicrographs showing identified features related to Samana Suk Formation of studied area (A) Red arrows showing blocky calcite vein and the replacement of bioclasts i.e. aragonite by calcite. (B) The yellow arrows showing the stylolite and slightly fractured surfaces. (C) The white arrows demonstrating towards the deformation of ooids, multiple enveloping concentric shells around ooids and pesoids simultaneously. (D) Green arrows pointing towards the muddy or lime groundmass of deep marine setting.

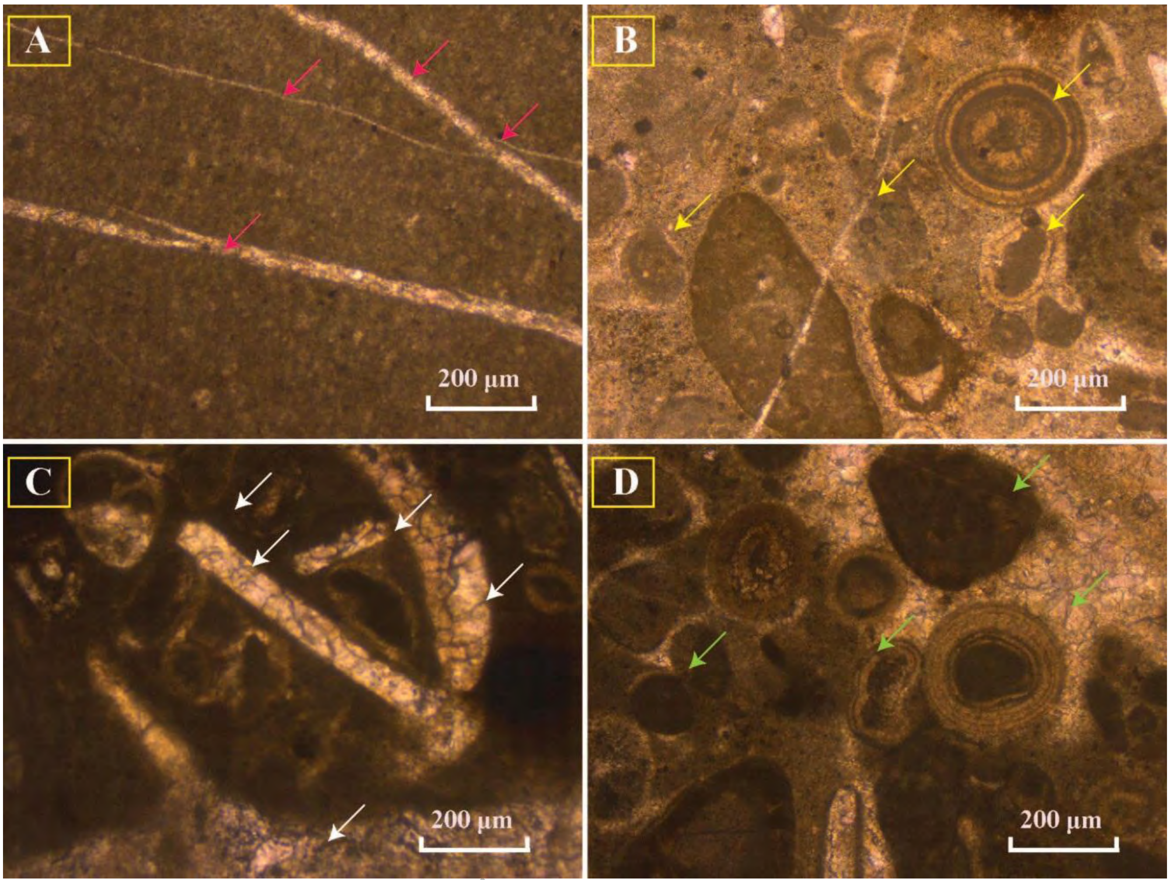


Figure: 4.2:- Photomicrographs showing the outcomes of studied area. (A) Pink arrows displaying the telogentic calcite veins. (B) Yellow arrows point towards the concentric shells of ooids. (C) White arrows shows the blocky calcite veins. (D) Green arrows shows the concentric shells of ooids and the process of micritization.

4.2 Field Tests Results

4.2.1 Exploratory Boreholes

To examine the subsurface rocks and soil practically as well as in laboratory drilling was performed on specific location where they were necessary to obtained the best possible results, for this total five boreholes were drilled with different depths, the maximum depth for a boreholes was kept 30m and the minimum depth was 15m. The diameter of rotary drilling was 76mm and for percussion machine it was specified at 250mm. The in-situ tests in the boreholes that were C.P.T (Cone Penetration Test) and S.P.T (Standard Penetration Test) performed in these boreholes at an interval of 1-1.5m and recorded. The results and detailed are given below.

Table: 4.1 Details of Drilled Boreholes

Sr.No	**Boreholes Designation**	**Required Drilling Depth (m)**	**Final Drilling Depth (m)**	**Structure Name**	**Remarks**	**CPT/SPT**
1	BH-01	30	10+10	On Left Abutment	10m w/Perc. & 10m w/Rotary	Refusal
2	BH-02	25	25	In Nullah	Percussion	Refusal
3	BH-03	30	30	On Right Abutment	Rotary	Refusal
4	BH-04	25	23	In Nullah	Percussion	Refusal
5	BH-05	15	15	Spillway	Rotary	Refusal

4.2.2 Field Permeability and Lugeon/Water Pressure Tests

Field Permeability tests were carried out by using constant head permeability test. Tests were conducted in five (05) number of Bore holes. The Lugeon test provides a degree of getting water by rock in-situ under pressure. These tests are basically very demanding to be used for the quantity of grout that rock will receive in order to check the efficiency of grouting and to get a measure of the quantity of fracturing of rock or to give an estimated value of the permeability of the rocks within there.

The easiness with which water can penetrate through a medium effortlessly is known as permeability of that particular soil within it. If a rock or soil is impermeable then it is obvious that it will have lesser amount of permeability or we can say negligible permeability and water will not pass through it unlike other permeable rock or soil which is why a totally impermeable soil does not permit the water to flow within it. Though, in nature such wholly impermeable soils don’t exist as they will permit some amount through pass within it, as all the soils are permeable to some extent. Table shows that clean gravel and coarse sands are very Good mediums for drainage, Fine sand and loose silt are classified as Fair mediums. While clayey silts, dense silt, silty clay and lean clay are Poor to very Poor medium and show low drainage properties. Strata in BH-01 has shown very permeable and very good drainage properties. Strata in BH-02 has shown Poor drainage properties, Strata in BH-03 has shown very poor drainage properties, strata in BH-04 has shown Fair drainage properties and strata in BH-05 has shown poor drainage properties with reference to (ASTM D 2434) Results are given below in table 4.2.

Table: 4.2 Permeability results from boreholes.

Sr.No	**Borehole**	**Coefficient of Permeability (cm/sec)**	**Drainage Properties**
1	01	1.1180×10^{-5}	Very Good
2	02	1.1178×10^{-4}	Poor
3	03	2.088×10^{-5}	Very Poor
4	04	1.52×10^{-3}	Fair
5	05	1.452×10^{-4}	Poor

4.2.3 Ground Water Observations

Ground water observations are the one of the most important parameter for all the structures and designs of major constructing projects co-related with ground that influence the formation and foundation. Not knowing the level and quantity of groundwater and constructing anything on the ground is very dangerous for the infrastructure, which is why a keen knowledge is required to study the quantity of groundwater table and the area itself. This is obvious if we have groundwater in subsurface that could affect the groundmass by several means like chemical active fluids may react with rocky strata if there or if we have soften rocks/soil that could be easily disintegrated and can bring lot of negative changes. Water in soils are often absorbed or can swell it up unlike the water in rocks that not only found in the cavities but they also fill the fissures. As we know that finer particles like clay and silt have a strong capacity to retain amount of water but the coarser grain particles like gravel and sand have less potential to absorb water and thus they are more permeable. In the studied area most of the strata < 5m were encountered as gravely and thus it has maximum permeability as compared to bedrock of reservoir which have strong competent hard strength rocks with minimum rate of permeability. Groundwater measurement method is dependent upon the scope of work. In our case Straight Rotary and Percussion drilling was performed and following water tables were encountered during our subsurface investigation to a maximum depth of 30 m in Month of January to March, 2022. The net results from the drilling are mentioned below in the table.

Table: 4.3 Details of water table at specific depth at each bore hole.

Sr.No	Boreholes Designation	Water Table (m)
1	01	Nil
2	02	3.0
3	03	12.0
4	04	3.5
5	05	Nil

4.2.4 Test Pits and Field Density Tests

For geotechnical investigations it is necessary that in the vicinity of area test pits and field density tests must be performed accordingly. The size of the pit will be such that a person can easily enter the pit, having a visual inspection and can carry out F.D.T (Field Density Test), for this a total of Sixteen (16) test pits were digged manually and the soil samples were get together from test pits for the purpose to analyze them. F.D.T as an acronym for field density tests. These are the important test that should be carried out while constructing any major project like highways, building foundation and dams ASTM (2008b). These tests were performed by sand cone method on selected seven samples in order to find out the exact compaction of layers the results are of all test pits are given below.

Table: 4.4 Detailed results of test pits and Field density tests.

Sr.No	**Test Designation**	**Field Density (g/cc)**	**Northing (m)**	**Easting (m)**
1	TP-01	1.600	3724844.802	612870.633
2	TP-02	1.880	3724760.673	612862.922
3	TP-03	2.360	3724769.245	612971.273
4	TP-04	2.020	3724875.353	612974.874
5	TP-05	1.350	3724834.066	613078.227
6	TP-06	1.480	3724531.246	613311.227
7	TP-07	1.352	3724569.428	613311.227

4.3 Geotechnical Characterization of Subsoil

4.3.1 Topography and Geology of the Site

Topography of the studied area mostly comprises of plain terrains and some mountainous belts near in the vicinity. However the specific location of the site is confirmed to be plain terrain. Observing at the site the furtherance of sedimentary rocks and soil units have been inspected, it is also noted that there was no impact of metamorphic or igneous activity. It is well-known that the huge conveyed boulders and cobbles of igneous rocks and metamorphic origin rocks are originate in the watercourse bedstead, emphasizing the existence of these rocks somewhere in the catchment space of Nainawarkhwar and possibly attached branches that have been transported by overflow of river and glacier.

Moreover the bed rock lithology of Baz Ali Dam comprises mostly of sandstone, shale and thin to thick bedded limestone. Looking closely to these rocks they are moderately to high weathered mostly covered at places the rocks exposures which give rise to stunted vegetation in the surrounding area, Furthermore the layers deposited on the surface of site are mostly comprised of Silt, clay and gravel with sand, poorly graded gravel, silty gravel, silt with sand, sandy lean clay and bedrock that is limestone. Moreover throughout these inquiries, the sub-surface was explored to an extreme depth of about 30 meters below the existing ground level (EGL) by doing drilling with rotary and percussion drilled parameters, as earlier mentioned that we have a total number of five boreholes, sixteen test pits and outcrop and boreholes sample for petrographic examination that were more enough for geotechnical and geological characterization of surface and subsurface sampling and analyzing the overall strata of the site. The drilled layer comprises a mixture of silty clay gravel, sand, poorly arranged gravel, silty gravel, and silt, sand and sandy lean clay, intervals of sand and limestone. The step by step encountering of subsurface strata in BH-01, the top layer encountered in bore holes was silty clayey gravel, sand and second layer encountered was silty gravel, sand and limestone. In BH-02, the top layer encountered was silty clayey gravel, sand and second layer was poorly graded gravel, sand and limestone. In BH-03, the top layer encountered was poorly graded gravel, second layer was silty gravel, sand, third layer was poorly graded gravel, sand, fourth layer was silty gravel and last layer was sand and limestone. In BH-04, the top layer encountered was silty clayey gravel with sand, second layer was clayey sand, gravel and third layer was sandy shale with limestone. In BH-05, the layers encountered was Light grey to brownish grey, clayey shale, moderately weathered, closely jointed, low strength and at the end with limestone beds.

4.4 Soil Parameters

After overall geological and geotechnical investigation during the field work and laboratory analyses, for the substrata, which would predominantly support the foundation loads, characteristic soil design parameters have been developed. These parameters are based upon the actual field data and engineering judgment i.e. after geotechnical investigations.

Diverse laboratory tests were accomplished on the samples that were engaged after doing all the necessary investigations in the field. Value were obtained on the basis subsoil parameters and adopted for bearing capacity. Design parameters for a particular depth of footing are given as;

Table: 4.5 Location; Left abutment at BH-01

Sr.No	**Depth (m)**	**Bulk Density (KN/m^3)**	**Unconfined Compression Strength (Kg/cm2)**	**Point Load Strength Index (MPa)**
1	0-3	---	---	---
2	3-4.5	1,920	200	---
3	4.5-6	---	---	---

Table: 4.6 Location; Nullah at BH-01 and 02

Sr.No	**Depth (m)**	**Bulk Density (KN/m^3)**	**Unconfined Compression Strength (Kg/cm2)**	**Point Load Strength Index(Mpa)**
1	0-4	---	---	---
2	4-8	1,800	160	---
4	11-16	1950	190	---
5	16-17	---	---	---
6	17-20	1980	240	---

Table: 4.7 Location; Right abutment at BH-03

Sr.No	Depth (m)	Bulk Density (KN/m^3)	Unconfined Compression Strength (Kg/cm2)	Point Load Strength Index (Mpa)
1	10-13	1850	2.0	---
2	18-21	2010	250	---
3	27-30	---	350	---

Table: 4.8 Location; Spillway at BH-05

Sr.No	Depth (m)	Bulk Density (KN/m^3)	Unconfined Compression Strength (Kg/cm2)	Point Load Strength Index (Mpa)
1	3-4.5	---	---	2.10
2	7.5-10	2035	---	---
3	10-15	---	375	2.70

4.5 Swell Potential

It is the amount of moisture or water content in the soil due to which the soil can swell. It is obvious from the definition of swell potential that the swell potential capacity is directly proportional to the amount of fine grain materials, as much finer the particle there will be greater chance of swelling in the post-constructed foundations. It can be directly determined in the laboratory by performing one dimensional swell test or indirectly from index properties such as plasticity index, clay content etc. Following method has been adopted in order to classify the soil as per their swell potential. Results can be seen in table 4.11.

Table: 4.9 The Kansas Highway Commission uses the Atterberg Limits (PI) to indicate potential expansive soils. In addition criteria for potential swell is below.

Sr.No	**Plastic Index %**	**Potential Swell Classification**
1	<15	Low or None
2	15-35	Moderate
3	>35	High

Table: 4.10 Dakshanamurthy and Raman uses modification of Plasticity Chart, which included PI and LL, with addition of the SL.A simplification of the procedure using LL is given below.

Sr.No	**Liquid Limit %**	**Potential Swell Classification**
1	0-20	Non Swelling
2	20-35	Low Swelling
3	35-50	Medium Swelling
4	50-70	High Swelling
5	70-90	Very High Swelling
6	>90	Extra High Swelling

Table: 4.11 Based on below table results for Swelling are categorized.

Sr.No	Sample#	Liquid Limit %	Plastic Limit %	Dakshanamurthy and Raman	Kansas Highway Commission
1	Sml-01	26.0	6.0	Non Swelling	Low to None
2	Sml-02	25.0	5.0	Non Swelling	Low to None
3	Sml-03	27.0	8.0	Non Swelling	Low to None
4	Sml-04	28.0	8.0	Non Swelling	Low to None
5	Sml-05	27.0	7.0	Non Swelling	Low to None
6	Sml-06	30.0	10.0	Non Swelling	Low to None
7	Sml-07	26.0	7.0	Non Swelling	Low to None
8	Sml-08	26.0	6.0	Non Swelling	Low to None
9	Sml-09	26.0	6.0	Non Swelling	Low to None

4.6 Laboratory Testing

For valuation of geotechnical, physical, engineering and chemical individualities, selection of the rock sample from boreholes, selection of rock samples from outcrop section and soil samples from test pits throughout our field in the vicinity of the studied area somewhere on right, left abutments and for construction materials studies respectively, these selected rock and soil samples were then equipped and exploited for further analysis in the laboratory. Petrographic analyses were carried out on rock samples collected from boreholes and outcrop sections opposite to dam body. Soil samples were examined in the laboratory rendering to the accepted laboratory testing program. The laboratory testing may vary conferring to the necessity and accessible material of the investigator. In this research the following tests were achieved; the laboratory testing was approved at testing facility of UET, Lahore and AKM Geotechnical, Lahore. The subsequent laboratory tests were accomplished on designated rock and soil samples respectively.

Boreholes and Test Pits

- Unconfined Compressive Strength Test
- Liquid limit and Plastic limit
- Point Load Strength Test
- Particle size distribution
- Maximum Dry Density and Optimum Moisture Content (M.D.D) and (O.M.C)

4.6.1 Unconfined Compression Test

To derive sample's elastic modulus and Poisson's ratio during the test, it is measured separately from the axial load, axial and lateral alteration. Undisturbed samples collected from boreholes as well as from outcrop section on particular area were subjected to determine qu (Unconfined compression) values of selected samples. Cohesion values (internal friction of rocks) were determined by following expression i; Cu=qu/2………………………………………………………………………..…. (i)

Following values were obtained from Unconfined Compressive Strength. The outcomes shows that the rock samples are of hard to very hard strength according to (ASTM D 1633), which are of high mechanical strengths.

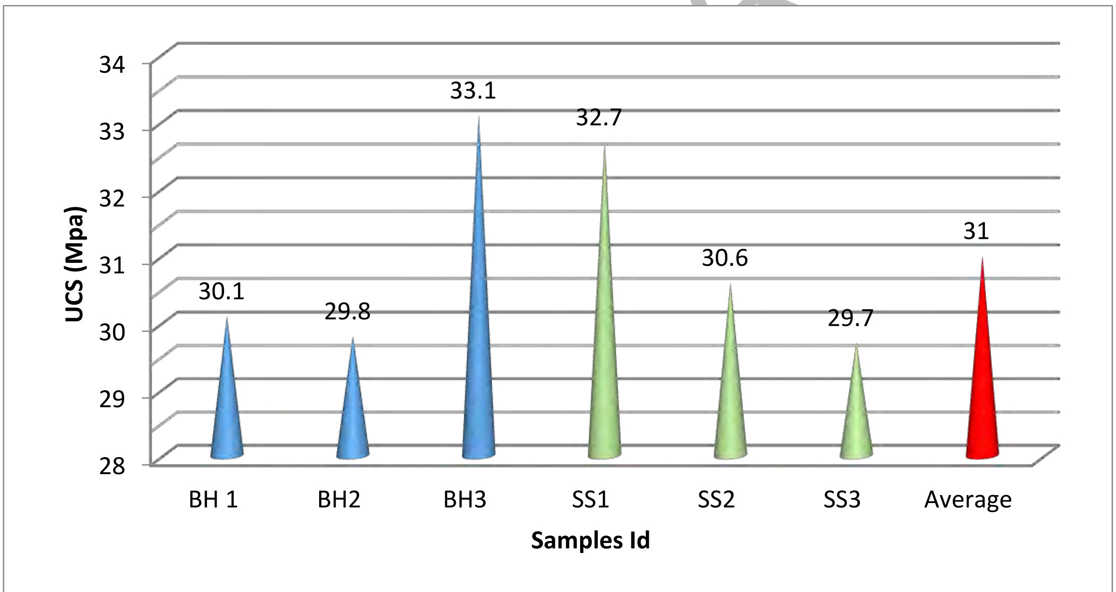
Table: 4.12 Descripted values for Unconfined Compression test from boreholes.

Sr.No	BH#	Sample#	Unconfined Compression Value	Cohesion
			Kg/cm^2	Kg/cm^2
1	1	CR-10	1.320	0.660
2	2	CR-14	390.45	195.2
3	3	CR-22	562.84	281.4

Table: 4.13 Descripted values for Unconfined Compressive strength test from outcrop section.

Sr.No	Outcrop#	Sample#	Unconfined Compression Value	Cohesion
			Kg/cm^2	Kg/cm^2
1	Left Abutment	SS-1	1.48	0.710
2	Right Abutment	SS-2	388.5	197.69
3	Spillway	SS-3	560.71	277.91

Table: 4.14 Comparison chart showing values from outcrop section with boreholes samples.



It is evident from the analysis that Qu values ranged from 388.5 Kg/cm2 to 562.84 Kg/cm2 with an average of 317.55 Kg/cm2 showing that rocky strata is hard to very hard, Showing strength class as very high will be helpful in bearing loads according to (ASTM D 2166).

4.6.2 Liquid Limit and Plastic Limit

These are essentially established on the basis of Atterberg's limits which comprise of the subsequent important values of moistness contents:

The moisture content at which a fine-grained soil can be elongated or drifts alike liquid is known as Liquid Limit. While the content at which a fine-grained soil can no longer be remolded without outrageous is called Plastic Limit. (Victor N. Kaliakin, in Soil Mechanics, 2017). Liquid Limit and Plastic Limit tests were accomplished on Twelve (12) particular rock samples from boreholes and ten (10) soil samples of prevailing sub-grade were verified to define the (L.L), (P.L) and plasticity index. All the liquid limit tests were implemented by means of at least three trials. Conferring to Casagrande's Plasticity Chart. The fined grained soils are ordered as CL-ML, CL (i.e. Silty clay and Low plastic Clay). The soil samples from test pits were found slight to little plastic. Test outcomes are delivered in table 4.15 and table 4.16.

Table: 4.15 Liquid limit and Plastic limit of different samples from boreholes.

Borehole #	**Sample #**	**Depth (m)**	**Atterberg's Limits**	
			LL%	PL%
BH-01	CPT-1	1.5	24.0	4.0
	CPT-2	3.0	25.0	5.0
BH-02	CPT-1	1.5	24.0	4.0
	CPT-3	4.5	25.0	4.0
	CPT-4	6.0	24.0	4.0
	CPT-6	9.0	26.0	6.0
BH-04	CPT-1	1.5	28.0	8.0
	CPT-2	3.0	29.0	9.0
	CPT-5	7.5	48.0	24.0
	CPT-6	9.0	50.0	25.0

Table: 4.16 Liquid limit and Plastic limit of different samples from test pits.

Sample#	Depth#	Atterberg's Limits	
		LL%	PL%
Sml-1	3.0	26.0	6.0
Sml-2	3.0	25.0	5.0
Sml-3	3.0	27.0	8.0
Sml-4	3.0	28.0	8.0
Sml-5	3.0	27.0	7.0
Sml-6	3.0	30.0	10.0
Sml-7	3.0	26.0	7.0
Sml-8	3.0	26.0	6.0

The over-all classification of the soil samples display they array amongst non-plastic to low-plastic. Similarly, the low plasticity soils have equitable shear strength although the highly plastic soils have poor shear strength. Thus, it is obvious that the organic soil has high plasticity values, low shear strength and consequently no appropriate for foundations. Shear strength regulates the workability of the soil as the lesser the shear strength, lesser the workability of the soil, although some features have to be measured. The difference in the proportion of clay and silt can be due to the change in the grade of weathering and the nature of the parent rock formation. Permitting to the arrangement of soil samples in Atterberg's limits are originate in the boreholes samples are given in the standard table 4.17.

Table: 4.17 Standard values for the Plastic Index

Sr.No	Plastic Index (%)	State of Plasticity
1	0	Non-plastic
2	1-5	Slight
3	5-10	Low
4	10-20	Medium
5	20-40	High
6	>40	Very High

4.6.3 Point Load Strength Test

In geotechnical practices point load strength tests are used to find out the rocks strength indexes. It thus enables the most economical testing of rock sample either in the field or in laboratory practices. For the calculation of rock strength it is an acceptable mechanical testing indexes. At a lower cost than U.C.S it can provide similar understandings as like U.C.S testing of rocks and thus it is used for more than 30 years in geotechnical practices (ISRM, 1985). The basic procedure for finding the rock mechanical strength with the help of point load testing involves the compression of rock's sample in between conical steel plates until failure of compressed rock occurs. It laterally consist of rigid frame two point load plates, hydraulically activated ram with measuring pressure and a device to measure the distance between two plates. The basic procedure for calculating point load strength indexes is established by the International Society of Rock Mechanics (ISRM 1985). The P.L.T is an effective technique to define complete rock strength possessions from drill core samples, Rusnak, J., & Mark, C. (2000). It has developed a recognized test in geotechnical assessments. Two (02) samples were prepared and subjected to P.L.T. The results are indicated in table 4.18.

Table: 4.18 Point load strength test of samples taken from borehole.

Sr.No	**Borehole#**	**Sample#**	**PL (Mpa)**
1	BH-05	CR-03	2.10
2	BH-05	CR-03	2.70

Table: 4.19 Results of point load strength tests outcrop samples.

Sr.No	**Outcrop#**	**Sample#**	**PL (Mpa)**
1	Right Abutment	SS-4	2.15
2	Left Abutment	SS-5	2.85

Table: 4.20 Standard values for point load strength and U.C.S.

Weathering Grade	**Schmidt Hammer Value L-9**	**Point Load Strength test ls (Mpa)**	**Uniaxial compressive strength (Mpa)**
1	34-40	2-3.2	42-63
2	22-36	1.2-2.1	26-45
3	14-24	0.4-1.5	11-28
4	12-15	0.26-0.55	3.5-12
5	<12	ND	ND

It is evident from the analysis that is range of Point load strength test (ls50) that is 2-3.2 showing the weathering grade coming in class I.

4.6.4 Particle size distribution (Sieve Analysis)

Sieve analyses were performed on sub-surface soils, Sixteen (16) designated soil samples from the boreholes and ten (10) selected soil samples from the test pits were put in to sieve analyses through these studies, the sieve analyses were accomplished in agreement with the techniques quantified in ASTM D-422, with sample preparation by ASTM D-2217 (wet sample preparation method).The proportions of fine sand and concretion segments of the tested soil samples are also specified in below table i.e. 4.21 and 4.22.

The dissimilarities in proportion of clay, silt can be owed to the dissimilarity in the amount of weathering and the environment of the photoliths. Permitting to the arrangement of soil and rock samples in BH-01 samples are specifically silty clayey gravel sand and 2nd layer is silty gravel sand, with limestone at the end, BH-02 samples are specifically silty clayey gravel, sand,2nd layer shown silty clayey sand, gravel, third layer was found to be Poorly graded gravel, sand and limestone, BH-03 samples are specifically poorly graded sand as top layer, 2nd layer is silty gravel, sand,3rd layer is poorly graded gravel, sand and limestone,4th layer was found to be silty gravel and also layers of silt, sand in alternate and the BH-04 samples are specifically Silty clayey gravel, sand,2nd layer is clayey sand , gravel. 3rd layer was found to be sandy lean clay.

Data obtained from test pits were showing the difference in the proportion of sand, gravel, clay with silt can be owing to the change in the point of weathering and the nature of the parent rock formation. Rendering to group of the soil samples at TP-3 are precisely A-2-4 which is silty or clayey sand with concretion, TP- 05 is specifically A-1-B which is Gravel and sand, SML-01 to SML-09 samples are definitely A-4,which designates that the important material is silty and quantity of clay.

Table: 4.21 Grain size analyses of samples from bore holes.

Borehole #	**Sample #**	**Depth (m)**	**Grain Size Analysis**		
			Gravel %	**Sand %**	**Fine Grained %**
BH-1	CPT-1	1.5	59.0	26.0	15.0
	CPT-2	3.0	44.0	26.0	30.0
	CPT-3	4.5	40.0	32.0	28.0
BH-2	CPT-1	1.5	77.0	16.0	7.0
	CPT-3	4.5	62.0	21.0	17.0
	CPT-4	6.0	62.0	25.0	13.0
	CPT-6	9.0	35.0	45.0	20.0
	CPT-9	13.5	79.0	20.0	1.0
BH-3	CPT-1	1.0	100.0	0.0	0.0
	CPT-2	2.0	58.0	26.0	19.0
	CPT-3	3.0	89.0	6.0	5.0
	CR-04	4.0	58.0	10.0	32.0
	CR-06	6.0	0.0	23.0	77.0
	CR-010	10.0			
	CR-14	14.0			
	CR-22	25.0			
BH-4	CPT-1	1.5	73.0	20.0	7.0
	CPT-2	3.0	22.0	49.0	29.0
	CPT-5	7.5	5.0	35.0	60.0
	CPT-6	9.0	0.0	30.0	70.0

Table: 4.22 Grain size analyses of samples from test pits.

Test Pit #	Depth	Sieve Analysis		
		10	40	200
	(feet)	pass %	pass %	pass %
TP-3	6.5	39.0	35.0	26.0
TP-5	6.5	63.0	26.0	11.0
Sml-01	3.0	2.0	15.0	83.0
Sml-02	3.0	2.0	19.0	79.0
Sml-03	3.0	6.0	24.0	70.0
Sml-04	3.0	8.0	30.0	62.0
Sml-05	3.0	0.0	25.0	75.0
Sml-06	3.0	4.0	24.0	72.0
Sml-07	3.0	4.0	17.0	79.0
Sml-08	3.0	0.0	22.0	78.0
Sml-09	3.0	-----	-----	-----

4.6.5 Maximum Dry Density and Optimum Moisture Content

To examine the M.D.D and O.M.C two (02) soil samples gotten from test pits were imperiled to their adjusted compaction tests. Results are delivered in table 4.23.

Table: 4.23 Maximum Dry Density and Optimum Moisture Content of the samples taken from test pits.

Sr.No	Test Pit	Max. Dry Density	Optimum Moisture Content (%)
1	TP-3	128.0	10.0
2	TP-5	128.0	10.0

The overall procedure to examine the M.D.D and O.M.C of soil samples are actually the compaction curve that is drawn amongst the water content at (X-axis) with corresponding dry density at (Y-axis), for this interpretations we have to understand primarily an increase in the water content with an increase in the dry density. Reduction of dry density is experimental in some way after it spreads at a specific point. The highest point of the soil compaction curve is when achieved it is called the maximum dry density rate of that substance. Water content resembles to this point is known as the optimum water content or optimum moisture content respectively. The association of the moisture-density with strength of the soil is founded on mechanical strength, with the increase in the numbers of blow there should be an upsurge in moisture content. Thus it is obvious that with the increase of number of blows there seemed to be increase in optimum moisture content. So with the increase in the number of blows beyond it limits will cause damage to the samples. Increase in moisture content due to increase in density; also there is increase in water content due to increase in the number of blows.

CONCLUSIONS

A detailed research work was accompanied to the planned area i.e. District Lower Kurram, Baz Ali dam. The leading subject of research work was to study the area geologically, geotechnically and to assess that the area is feasible for dam construction. The diverse geo-technical evaluations conceded out on the samples (disturbed and undisturbed) from the location, to conclude the competency of rocks. The conclusion of current research work is summarized as;

- The Unconfined Compressive Strength and other mechanical properties showed moderate inverse relationship with sparite and allochems, whereas micrite and limestone indicate a positive correlation with U.C.S with respect to petrographic analyses, the process of micritization take place in the rocks valuing the competency of rocks within there.
- There were no such major structural discontinuities that could affect the infrastructure of reservoir.
- The Atterberg's end result displays state of plasticity for soils range from 'Slight- Low-Medium' which also matches Casagrande's arrangement which displays that maximum amount of inorganic soils with low plasticity.
- Soil samples having very low permeability which categorizes them as impermeable soil, which is a significant limitation for the appropriateness of construction.
- Shear strength of the rocks displays reliability of the samples as 'very hard'.
- The Sml samples collected, in order to check the availability of construction of material nearby vicinity has shown the presence of A-4 material. As per AASHTO classification which can be used as construction material.

RECOMMENDATIONS

In addition to spelling-out the foundation design parameters, foundation type, depth and allowable bearing capacity, the following important recommendations were made for construction of foundation because of the peculiar site conditions:

From petrographic and fieldwork analyses it is recommended that some parts of the abutments that were dolomitized should be grout and slope should be accomplished. The digging through construction process may be prepared at a slope as stated earlier.

Competence of the basis depth should also be tested by safeguarding proper influences of protection against uplift and adjacent loads.

Further dams can be built in the vicinity.

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