Vol. 3, No. 2 (2022); 126–134

Journal of Earth Sciences and Technology

http://www.htpub.org/Journal-Of-Earth-Sciences-And-Technology/

Geotechnical Evaluation for the Residential Blocks in Bara Kahu Region, Lesser Himalayas, Pakistan

Syed Muhammad Hassan[a]*, Mian Sohail Akram[a], Luqman Ahmed[a], Muzamil Hussain[a], Irfan Ghani[a]

[a]*Institue of Geology, University of the Punjab, Quaid-e-Azam Campus, 54590 Lahore, Pakistan*

Article Information

Article History
Received: 11/04/2022
Accepted: 01/05/2022
Available online: 25/07/2022

Keywords
Limit Equilibrium Method
Slope Stability
Residential Blocks
Cut Slope
Support

Abstract

Cities are extending, and the increasing population demands the development of new societies and residential blocks. The shortage of places in the plain moved the constructor to construct residential blocks in hilly and mountains. Developing such blocks requires detailed geotechnical studies and evaluation of the slopes that need to be cut to place the structures on them. In the current research, the geotechnical evaluation for the slopes along with the residential blocks in Bara Kahu. In this research, field data comprising discontinuity parameters such as spacing, persistence, aperture, roughness, block size were determined from the surface rock exposures along the natural slopes. The limit equilibrium analyses were conducted on both natural and cut slopes for the assessment of Factor of safety. The batter cut was determined from these analyses for the residential blocks.

1. Introduction

The designing of the slope with proper investigations and evaluation of rock masses is always recommended by various researchers (Stille & Palmström, 2003). Cutting the slope without proper planning can cause different failures such as plane, wedge, and toppling failures. Rock and soil slope classification is partially based on failure modes (Akram et al., 2018, 2019; Noor et al., 2017a). In rock mass, slope failures are governed by the pattern of discontinuities. Plane failure occurs along the predominant discontinuities that dip in the direction of the slope. In comparison, wedge failure occurs along the intersection of two different planes of discontinuities. On the other hand, toppling failure occurs along the discontinuities that dip opposite the slope (Markland, 1972, 1974; Wyllie & Mah, 2004). Rock slope stability analyses are usually carried out by empirical methods, limit equilibrium or numerical simulations (Admassu & Shakoor, 2013; Hussain et al., 2015; Li et al., 2009; Majeed & Bakar, 2015). The classification of rock mass for the evaluation of rock-cut slope is based on the most important

* Corresponding author
E-mail address: mhassan.mphil.geo@pu.edu.pk

structural parameters. For designing the slope, different parameters are used to calculate the quality of rock mass. These parameters are part of renowned classifications such as Rock Mass Rating (RMR) (Bieniawski, 1989, 1976; Celada et al., 2014), Slope Mass Rating (SMR) (Ahmad et al., 2017), and Geological Strength index (GSI) (Hoek & Brown, 1997). RMR is a mathematical sum of different parameters. The RMR classification is sole as a slope design method, and SMR is used to measure slope stability by incorporating the failure modes (Akram et al., 2018; Mondal et al., 2016; Romana, 1993; Romana et al., 2015). Back analysis of failed slope is the most reliable method of determining the rock mass strength (Frayssines & Hantz, 2009; Noor et al., 2017b). SMR is based on the RMR system are depending upon the jointing conditions and excavation methods, respectively.

For stability of rock slope, two methods should be used side by side. Limit Equilibrium methods is used for calculating the Factor of safety of slope based on failure criterion. The limit equilibrium method is based on the method of slices (Bishop, 1955; Janbu, 1954; E Spencer, 1967). For example, the ordinary method of slices and the Bishop simplified (Bishop, 1955), Janbu simplified (Janbu, 1954), and Spencer method (E Spencer, 1967; Eric Spencer, 1969) are the methods of slices to define the factor of the slope's safety. These methods identified protentional failure mechanisms and drives factors of safety for a particular geotechnical slope.

According to the limit equilibrium approach, in limiting condition factor of safety value equal to 1, when the value is greater than 1 the slope is stable and in an unstable slope the limit equilibrium value is less than 1. The desired value of factor of safety depends upon the importance of the slope and the consequences of failure. One of the important input parameters of the limit equilibrium analysis is the geological strength index (GSI). The original Geological Strength Index (GSI) chart was developed on the assumption that observations of the rock mass would be made by qualified and experienced geologists or engineering geologists. When such individuals are available, the use of the GSI charts based on the descriptive categories of rock mass structure and discontinuity surface conditions are used for empirical rock mass classification. The GSI chart published by Marinos et al. (2005). GSI is generally assessed based on the rock structure (number of discontinuities sets and spacing) and surface conditions of the discontinuities (i.e., aperture, infilling, wall roughness, and weathering grades) regardless of the geologic origin. GSI ranges from 1 to 100 such as one corresponds to very poor quality of rock, that is, highly fractured, sheared, and seamy, having numerous discontinuities. Whereas 100 represents a very good quality rock mass devoid of discontinuities, just like intact rock. In the current research, the limit equilibrium analyses were carried out to find out the most suitable cut slope angles for the residential blocks.

2. Geology of the Study Area

The present research is carried out to evaluate the better slope cut for the placement of residential blocks in Bara Kahu, Pakistan. The study area lies in the sub-Himalayas. The Project area Geology is covered by unconsolidated deposits and rock units. Unconsolidated deposits are mainly comprised of colluvial, which is gathered at the toe of the slope face. The rock exposed to Murree Formation, whose age is Miocene which comprises the alternative beds of competent and incompetent lithologies. Competent lithologies are sandstone and siltstone, while incompetent lithologies are claystone, mudstone, and shales. The project area lies in the Zone 3, which is the seismically active zone, the peak ground acceleration (PGA) is 0.24-0.32g (BCP, 2007). The incompetent lithologies claystone/mudstone are thinly to thickly bedded very low to low persistence moderately to highly weathered, and shales are the weakest lithologies which are comprise thinly to thickly bedded with low to very low strength. At some adjacent localities in the project area, the cut fill has covered the geological exposures. In the covered area extrapolating the recorded orientation of various rock beds were exposed and recent subsurface investigations for foundation design have helped interpret the surface and subsurface geologic condition below the fill. In some place’s shales, claystone and sandstone are reddish-brown color in most part of project area, purple to grey to greyish-brown sandstone, siltstone with the subordinate bed of reddish-brown to brown mudstone, shales beds are present.

The project area lies in a territory characterized by intensive structural disturbance. The main lineament 8~10 km in the north and northwest of the project area is northeast-southwest trending Main Boundary Thrust (MBT) dipping northward. In addition to MBT, many thrust faults run parallel to the MBT and thrust opposite MBT, i.e., southeast. Tectonically the region is an intensely deformed belt, which along with the Attock-Cherat Range and the Margalla Hills, represents the uplifted southern margin of the Peshawar Basin. It is part of the active Himalayan foreland - fold and thrust belt region in the collision zone between Indo-Pakistani and Eurasian plates (Kazmi & Jan, 1997; Shah, 2009). The area southeast of MBT is characterized by an undulation-making series of anticlines and synclines. The undulation has a general trend parallel to MBT and other thrust faults, i.e., northeast southwest. The project area lies on the western limb of a syncline having plunge northward.

In most of the study area, the rock outcrop is under the thin to the thick cover of overburden soil comprising reddish-brown clay, silts, with some rock fragments. However, recent cut and fill activity has obscured the virgin area under the fill derived from cutting, as per project requirements. The rocks exposed belong to the Murree Formation of early Miocene age that comprise alternate beds of competent sandstone/ siltstone, with incompetent shales/mudstones and claystone with thin layers of silts and clays at places. The orientation of rock units, as recorded at the site, is NE-SW trending strike with dip variation from 29~85 degrees in NW direction also have been marked on site geologic plan (Figure 1). However, variation in the strike of vertical joints from NE-SW to NE-SE direction was also observed at some localities. Orientation of various discontinuities/ joint sets, where possible, was also recorded to have an idea of the general orientation of the deformation. The recorded joints were mainly observed in sandstone/siltstone beds and are marked on the site geologic plan.

At some adjacent localities in the project area, the cut fill has covered the geological exposures. For the covered areas, extrapolating the recorded orientation of various rock beds (where exposed) and recent subsurface investigations for foundation design have helped interpret the surface and subsurface geologic conditions below the fill.

As part of the surface geology investigations and evaluations, the area is evaluated with five (5) Discontinuity Surveys (DS). The locations of conducted DS have been given in Figure 1. According to the surface geology, the sandstone/siltstone beds, mostly grey to greyish brown, are exposed with interbedded shales/ claystone and mudstone. The thickness of the sandstone/siltstone beds varies from 15 ft. to 70 ft. The sandstone/siltstone is thin to thick-bedded, low to medium persistence, slightly to moderately weathered, fine to medium-grained, compacted, weak to strong, medium-hard, and closely fractured/ jointed. The persistence of joints generally runs across the sandstone beds.

On the other hand, in the incompetent lithologies, claystone/mudstone are very thinly to thin bedded, at places thickly bedded, very low to low persistence, moderately to highly weathered, less compacted, very weak to weak, intermediate slaking to swelling, and very jointed. Among the incompetent units, shales are the weakest lithology. It is splintery, slaking, and of swelling type, very weak to weak by strength, very thin to thinly bedded, moderately to highly weathered, at places completely weathered were observed in the project area. Besides shale/claystone and sandstone, some layers of reddish-brown silts/clays have also been reported.

3. Methodology

A comprehensive methodology was purposed for the present research that comprises field investigation, geological Mapping, laboratory testing and limit equilibrium methods. During the field investigations discontinuities surveys were performed on competent and in competent units similarly the soil and rock samples are gathered for the laboratory testing. A total of five discontinuity surveys were conducted together with recording spot reading of orientation in the research area. The locations of the discontinuity surveys have been marked on the geological map as shown in the Figure 1. Detail of discontinuity surveys comprising locations, coordinates and rock type are provided in Table 1. Schmidt Rebound hammer was used for the assessment of strength, according to the guidelines of international society of rock mechanics (ISRM [International Society for Rock Mechanics], 1981). The value of rebound number (Rn) was calculated by the correlation chart of unit weight. Geological Strength Index (GSI) (Marinos et al., 2005) was calculated by the correlation is given below:

$$GSI=RMR-5. \tag{1}$$

Table 1. Summary of conducted discontinuity surveys.

DS Location/ Sample No	Easting	Northing	Identified Rock
DS-1	334028	3732814	Claystone/Mudstone
DS-2	333976	3732768	Sandstone/Siltstone
DS-3	334333	3732622	Sandstone/Siltstone
DS-4	333963	3732653	Sandstone/Siltstone
DS-5	333867	3732586	Sandstone/Siltstone

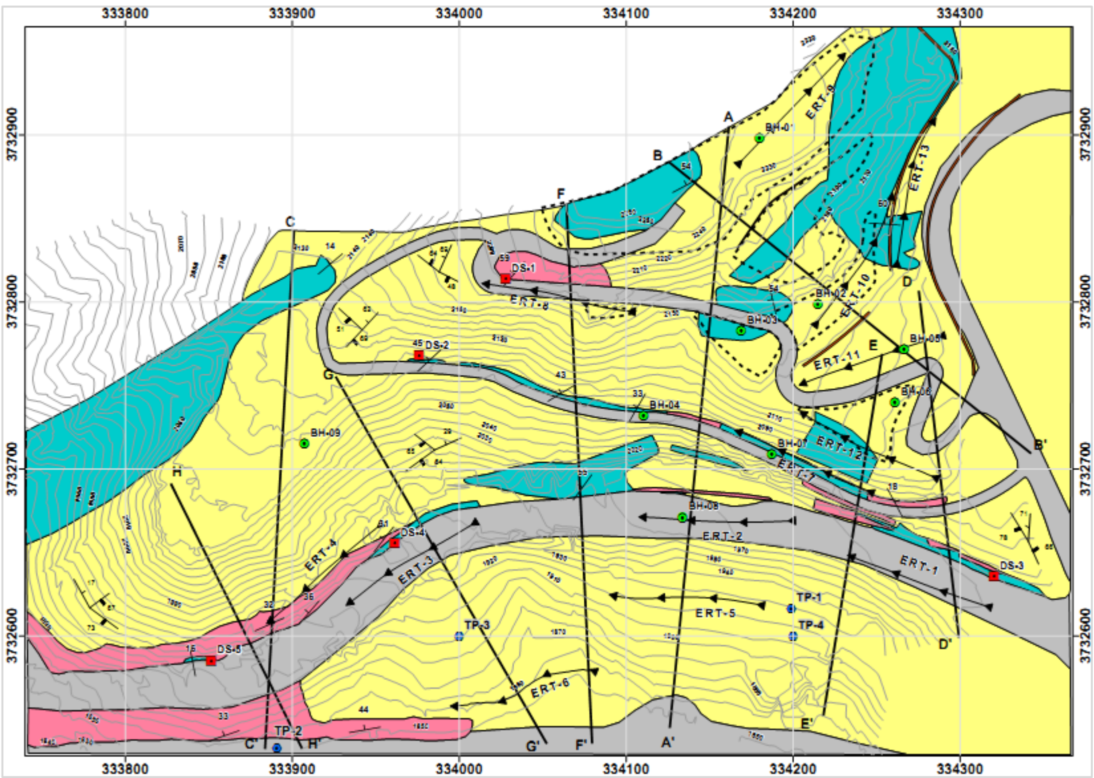

Fig. 1. The geological map of the research area.

Procedure of evaluating and predicting the possible direction of moment of rock is called as Kinematic analysis. Kinematic analysis is a method in the laboratory, unit weight, density, point load test were performed. The Barton Bandis failure criteria that use joint compressive strength (JCS) and joint roughness coefficient (JRC) were utilized in the slide analysis. Soil and rock parameters were derived based on the laboratory test results, field studies and testing, engineering judgment, and recent literature. Intact rock mechanical properties were determined by analyzing the laboratory test data. The laboratory tests results of samples have been analyzed to adopt the intact rock parameters. The data includes the point load strength index (Is50), uniaxial compressive strength (UCS), unit weight, and water absorption. In sandstone/siltstone and claystone/mudstone, the average value of 160.43 lb/ft3 and 153.36 lb/ft3 were observed as unit weight. The uniaxial compressive strength (UCS) and Point Load Strength Index (Is50) were measured in the laboratory. The estimation based on the point load test ranges from 1.06 to 9.07 MPa with a mean of 5.07 MPa for sandstone/siltstone, and for claystone/mudstone, the point load values range from 0.16 to 5.87 MPa. The UCS based on the UCS testing range from 13.52 MPa to 58.93 MPa with a mean of 30.85 MPa for sandstone/siltstone. For claystone/mudstone, the values range from 3.04 to 32.85 MPa. The summary of the adopted parameters for slide analyses are given in table 2.

Table 2. Summary of intact rock parameters based on laboratory test data.

Rock Type	Sample ID	Uniaxial Compressive strength (PSF)	Point Load Index Strength (PSF)	Unit Weight (lb/ft^3)
Sandstone/Siltstone	Minimum	282371	22053	155.05
	Maximum	1230780	189431	163.97
	Mean	644316	105955	160.43
	Adopted Values	644000	105000	160
Claystone/Mudstone	Minimum	63492	3416	143.93
	Maximum	686087	122633	160.63
	Mean	397137	25400	153.36
	Adopted Values	208854	20000	150

As per the prevailing practices, the following combination of loading conditions and correspondingly required safety (FOS) factors have been adopted for this evaluation (Table 3).

Table 3. Factor of safety versus loading conditions.

Slope Type	Loading Conditions	Factor of Safety
Cut slope	Long term (Steady-state seepage)	> 1.5
	Short term (Fully Saturated)	> 1.2
	Extreme (Steady-state seepage with earthquake)	> 1.1

4. Results and Discussion

Firstly, the natural slope was modeled and evaluated to examine whether the adopted parameters of the model were realistic scenarios using the adopted parameters. In the first run, the natural slope was modeled in existing conditions without earthquake loading. The factor of safety (FOS) obtained by various methods and all sections is greater than 1.5.

Afterward, the models were provided with the earthquake loading, and a FOS of greater than 1.1 was determined, indicating it as stable in the existing geometry and conditions against earthquake loading. Since the natural slope has experienced many earthquakes of greater magnitude and stayed stable, the modeled scenario with earthquake loading presents a realistic picture. The factor of safety with the Bishop method of slices is given in table 4. The parameters are reasonable and can be used for further analysis in rock-cut slopes.

Table 4. FOS for Natural Slope.

Sections	The factor of safety (FOS)			
	Normal Condition	With Seismic Loading	With Flooded Condition	Extreme Conditions
AA'	2.0	1.6	2.0	1.4
EE'	1.8	1.3	1.8	1.3
FF'	1.8	1.3	1.8	1.3
GG'	1.5	1.1	1.5	1.1
HH'	1.7	1.3	1.7	1.3

In the slide analysis and various analysis were performed at different cutting angle of 4V:1H, 3V: IH and 2.5V:1H. These analyses were performed in four conditions those are normal, seismic condition, flooded condition, and extreme conditions. The width of the bench is set at 10 feet, and the slopes have been evaluated with and without reinforcement. In the global stability analyses, the natural slopes were found stable in extreme conditions, however, cut slopes evaluated at angles 4V:1H, 3V:1h and 2.5V:1H having 10ft wide benches after every 30ft slope height necessitated the need of support for stabilization. The model prepared in the slide is shown in figure 2.

The slope geometry has been modified to include the rock cutting angle of 4V:1H, 3V:1H, and 2.5V:1H were used, with the provision of 25-30ft height and 10ft wide benches. Also, at the locations of the access road placement, the bench width was kept about 130ft, and the width of the main road of 170ft is used. For the placement of the house, the 100ft width of the plot is used. The models were run with the geomaterial properties adopted for the natural slope analyses with three various cutting slope angles, those are (a) rock cut slope at 4V:1H, (b) rock cut slope at 3V:1H and (c) rock cut slope at 2.5V:1H.

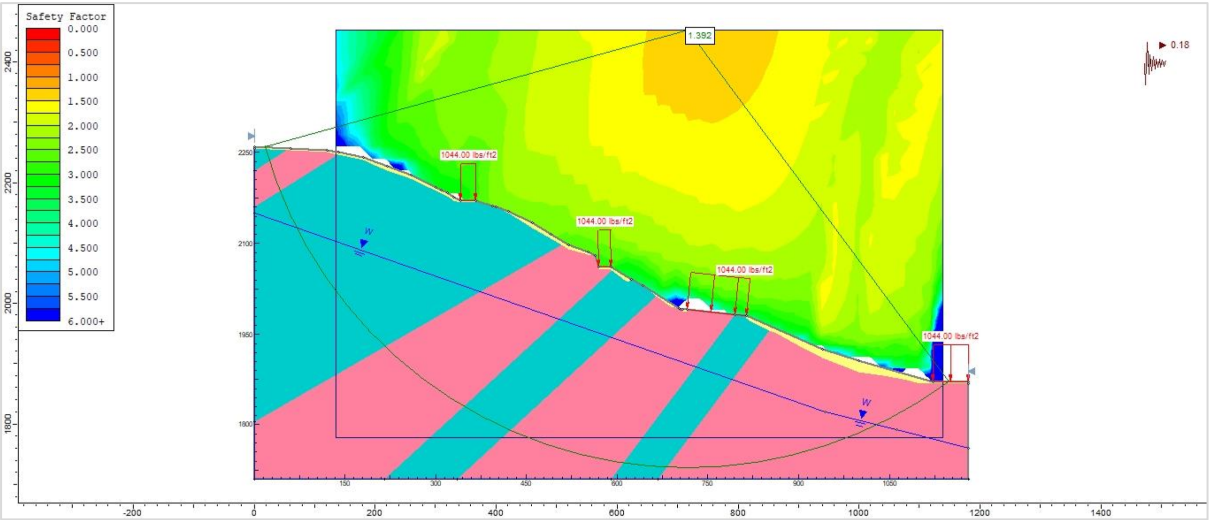

Fig. 2. Results of Limit Equilibrium Analysis for 2.5V:1H in extreme conditions.

The cut slopes analyses are now run with support. The support elements are mainly the rock bolts (grouted tieback) with a length of 15ft spaced at 10ft. The bolt properties are summarized in table 5, while determined FOSs for the section A-A' are provided in

Table 6. The model of cut slopes is given in figure 3.

Table 5. Support properties used in the analysis.

Support Parameters	
Tensile Capacity (lb)	38200
Plate Capacity (lb)	22480
Bond Strength (lb/ft)	110630
Bond Length (%)	100

Table 6. Details of the slope geometry, provided support, and determined factors of safety.

Cut-slope details			Support details		Factor of Safety (FOS)		
Cut Angle	Bench width (ft)	Bench height (ft)	Rock bolt length	Spacing	Bishop	Spencer	Janbu
4V:1H			NIL		0.54	0.57	0.50
4V:1H			15ft	10ft (systematic)	1.05	1.09	0.99
3V:1H	10	25-30	NIL		0.59	0.69	0.47
3V:1H			15ft	10ft (systematic)	1.15	1.14	1.08
2.5V:1H			NIL		0.64	0.66	0.63

2.5V:1H	15ft	10ft (systematic)	1.25	1.23	1.15

(a) (b)

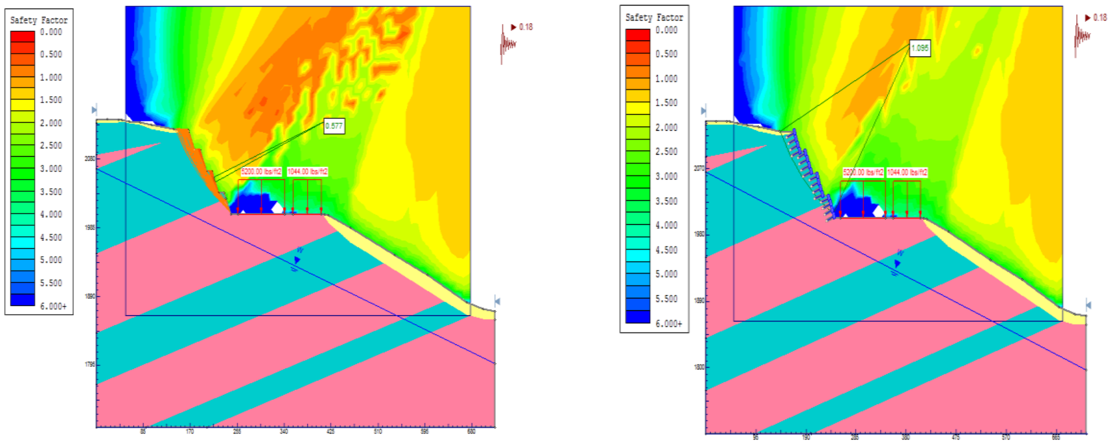

Fig. 3. The cut slope models for the rock slopes 4V:1H (a) without support (b) with support.

5. Conclusions and Recommendations

The modelled and recommended support comprises 15ft grouted bolts at rectangular spacing of 8-10ft. In addition to rock bolts, the cut slopes will be provided with 10cm thick shotcrete with wire-mesh in two layers with wire-mesh. The shotcrete will be hold intact by the application of rock dowels having 5ft length such that 3-3.5ft in the ground and rest bending against the wire mesh. To provide steady flow conditions and keep the water table, if any, away from the determined critical surfaces, necessary drainage comprising weep holes up to minimum of 15ft depth in to the rock shall be provided at a suitable spacing (8-10ft) to avoid building-up pore water pressure along the likely critical surfaces and behind shotcrete. Factor of safety (FOS) plays an important role in the slope stabilization methods. We calculate factor of safety at different stages such as Normal condition, with Seismic Loading, with Flooded condition and Extreme Conditions. In normal condition factor of safety should be greater than 1.5 and in extreme condition factor of safety should be greater than 1.1. In this research there are five cross-sections along the slope AA', EE', FF', GG' and HH'. We cut the slope along these sections and calculate the factor of safety of the slope. In all these sections we observed that, in an extreme conditions factor of safety greater than 1.1.

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