Sci.Int.(Lahore),28(6),5111-5114,2016 ISSN: 1013-5316; CODEN: SINTE 8

GEOTECHNICAL ASSESSMENT FOR WEIR FOUNDATION INCORPORATING ROCK MASS RATING (RMR) AND GEOLOGICAL STRENGTH INDEX (GSI) CLASSIFICATION SYSTEMS OF A SMALL HYDROPOWER PROJECT, DISTRICT SHANGLA-PAKISTAN

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ABSTRACT**:** *Borehole data was used to classify and characterize the rock mass by using empirical classification systems, Rock Mass Rating (RMR) and Geological Strength Index (GSI). GSI of each classified rock unit was assessed to derive rock mass parameters using RocLab (a computer program). Using rock mass parameters allowable bearing capacities of all zones encountered in boreholes were determined by three well-known methods: Terzaghi equation for general shear failure, bearing capacity factors by Bell solution and equation of compressive failure. According to the RMR rating values (21-37) the rock mass categorized as a poor rock and fall in class IV. According to the GSI rating values (39-58), the rock mass is blocky to very blocky. The results have shown that the allowable bearing capacity is 2.31-4.03MPa (compressive failure equation) and this equation is favorable with respect to geological conditions of the proposed weir.*

Keywords: Rock Mass Rating (RMR), Geological Strength Index (GSI), Allowable bearing capacity equation

INTRODUCTON

The failure of rock may occur under applied load due to rock mass properties. Foundation design for important structures weir etc. depends on the accuracy of bearing capacity estimation of underlying rock [1]. Stability of weir foundation has great importance both for safety and economics of the construction impact. So, it is important to determine the bearing capacity of foundation [2]. In view of intact rock properties and discontinuity characteristics, characterization and classification of the rock mass has been undertaken using empirical rock mass classification systems like RMR (After Bieniawski [3] and GSI [4]. Ajjotheri [5] studied the effect of aperture, spacing, density and persistence of discontinuities on rock mass. These rock mass parameters used in RMR system for support assessment to stabilize the rock mass.

The safety of the weir site is very important, so in this study bearing capacity of subsurface rock is calculated by different equations and these bearing capacity values compared with RMR and GSI of the same subsurface rock. Rock mass characterization has significant importance for design of dams or hydropower structures.

Hydropower project is a major structure for generation of electric power. Now a days our country is facing a huge shortage of electricity. Government is intended to build small hydropower projects to mitigate the electrical shortage. For this purpose number of small hydropower projects has started in Northern area of Pakistan. The main focus of the engineers is on the stability of weir site of hydropower. For above mentioned requirement of electricity a small hydropower project in Shangla district is selected for the present study which is situated along Besham- Swat road.

By using three boreholes data RMR and GSI calculated for rock mass classification and bearing capacities at different depth are also calculated for the assessment of weir foundation. Correlation between rock mass classification systems and calculated bearing capacities is developed through correlation chart that shows the trend of change in rock strength.

GEOLOGICAL SETTING

The Geology of the study area has been interpreted based on the Geological map of 43B (Degree Sheet, Scale 1:250, 000) compiled by Geological Survey of Pakistan (GSP), Ministry of Petroleum, Pakistan. The regional geology of the study area is presented in Fig. 1 with following geological units: Karora group, Besham group and Gandaf Formation.

Karora group

The Karora group is composed of granitic schist, dark fine grained quartz mica schist, minor tremolite marble, calcite, quartz, dark and fine grained metapsammite. Fletcher et al. [6] used ''KaroraFormation'' for marine metasediments which was unconformably deposited over the Besham group.

Besham group

Besham group is dominantly composed of quartzo-feldspathic gneiss, granitic gneiss, graphitic schists, and minor quartzite. The gneisses are light grey, medium grained and equigranular [7]. This group also contains Pelitic metasediments. They include very fine to medium grained graphitic schists and mica schists. The upper contact of Besham group with Karora group is unconformable. The Besham group is composed of dark biotite granitic gneiss, granodiorite gneiss, Lahor leucogranite, biotite orthogneiss, leucogneiss, pegmatite, schist, marble and mafic intrusion.

Gandaf Formation

Khan et al. [8] used the name Gandaf. The lower contact of Gandaf formation is with Karora formation. Gandaf formation is composed of graphitic slate, Phyllite, marble mafic intrusions Quartzite, fine grained metapsammite, argillite, calcite marble, tremolite marble, quartz schist, garnet mica schist, pegmatite, leucogranite, biotite gneiss

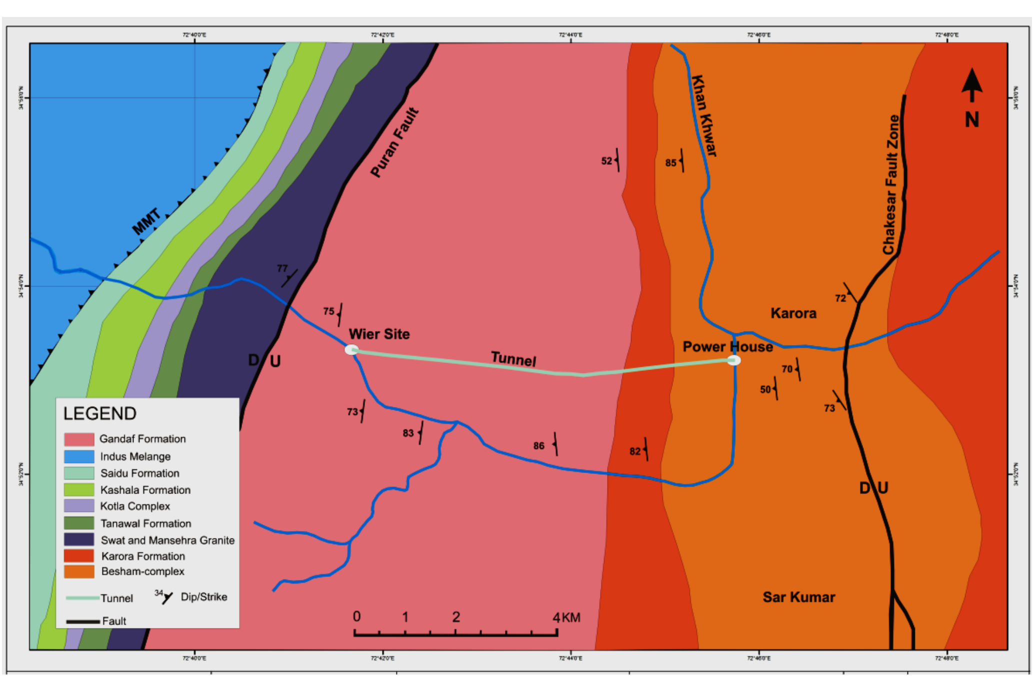

Fig. 1. Regional geological map of the study area.

The research area is characterized by steep mountains. Almost at the mouth of all tributaries, fan deposits are present that has been deposited in ancient times and are under cultivation and settlements. Almost all main villages are on these fans and terraces. According to map, study area situated in Gandaf Formation having age of Early to Late Proterozoic and it was deposited on the Karora Formation comprising graphitic, garnet schist, graphite slate, phyllite and schist, fine grained metapsammite, argillite, calcite marble, tremolite marble and quartzite. In the project area, the exposure of Gandaf formation comprise alternate beds of quartzite and amphibolite, schists and alternated beds of marble and quartzite. The field studies have revealed that the rock formation mainly comprise alternate beds of quartzite and amphibolite having trend almost across the river (N6-16E) with mainly dip in downstream (i.e. 74-88SE) and upstream (i.e. 69-86NW) direction at places. Two joint sets having orientation of (J-1=N3-81E, 17-84NW, J-2=N78-86W, 59-64NE) were also observed with some random joints during scanline surveys at weir site.

The left and right abutment of the weir occupied by terrace deposits. These terrace deposits can be categorized into two types; lower terrace deposits present at lower levels on the banks of river and upper terrace deposits present at higher levels on the slopes and valley. Lower terraces generally consist of the old river deposits and are covered with the silty / clayey material deposited by the river or from slope washes. These consist of light gray to gray colored sandy gravels with varying sizes of coarse fractions of angular, rounded to sub-rounded cobbles and boulders, strong to very strong in nature. Whereas upper terraces generally consist of both the alluvial and the colluvial (slope wash) material. These normally possess thin to thick cover of silty / clayey material and are under agriculture use or settlements. The terrace material is light gray to yellowish brown, firm to stiff, silty clay / clayey silt, overlying the material of varying size from gravel to boulders and occasionally of rock blocks.

The river bed material consists of the gray, loose, sandy gravel cobble with boulders generally of rounded to sub rounded and strong to very strong in nature. This material is of metamorphic and igneous origin. The sand present as matrix is fine to medium and coarse grained in places, and is micaceous in nature.

MATERIALS AND METHODS

Present investigation included three boreholes up to maximum depth of 25m below natural surface level were proposed on the weir site for the evaluation of allowable bearing pressure for the encountered rock mass. Drilling boreholes were distributed as one hole on each abutment and one in the valley (river bed). The holes were cored using NX standard size. Afterwards, geotechnical logging of the borehole from the retrieved core samples was undertaken to account for the rock type, physical properties (color, grain size, mineralization, etc), estimation of rock strength, and characteristic of the discontinuities.

Numeric ratings are assigned for rock mass quality by using rock mas classification systems like RMR and GSI [9]. These systems are based on easily measurable rock and discontinuity parameters and are used to provide an estimate of support requirements, strength and deformation properties of the rock mass [10].

RMR is suitable system for support design in tunnels, slopes and foundations [11]. This system has been refined over the years due to a better understanding of the importance of the different parameters [10]. RMR system has six parameters (Uniaxial compressive strength, Rock Quality Designation, Spacing of discontinuities, Condition of discontinuities, Ground water conditions, Orientation of discontinuities) that are used to classify a rock mass [2,3].

The GSI was introduced to overcome the deficiencies in Bieniawski's RMR system. It provides a system for estimating the reduction in rock mass strength for different geological conditions as identified by field observations. Quantitative chart is used for this study (After Sonmez, et al. [4]. GSI value is obtained by combining the two parameters, one is structure rating (SR) based on volumetric joint count (Jv) and other is surface condition rating (SCR) estimated from sum of three parameters, they are roughness, weathering and infilling materials.

There are two terms used for bearing capacity, ultimate bearing capacity and allowable bearing capacity [12]. In literature number of bearing capacity equations are reported for the calculation of ultimate bearing capacity. Following equations are more relevant to the rock mass conditions of the study area.

Terzaghi's Equation for general shear failure

The ultimate bearing capacity for the general shear mode of failure can be estimated from the Buisman-Terzaghi (1943) bearing capacity expression as defined by this equation [13].

$$q_{ult} = cN_c + 0.5\,\gamma B N_\gamma + \gamma D N_q$$

$q_{ult} =$ The ultimate bearing capacity

γ= Effective unit weight (i.e. submerged unit wt. if below water table) of the rock mass

$B =$ Width of foundation

$D =$ Depth of foundation below ground surface

$c =$ The cohesion intercepts for the rock mass

The terms N_c, N_γ and N_q are bearing capacity factors given by the following equations.

$$N_c = 2N\text{ø}^{1/2}(N\text{ø} + 1)$$

$$N_\gamma = N\text{ø}^{\frac{1}{2}}(N_\text{ø}^2 - 1)$$

$$N_q = N_\text{ø}^2$$

$$N_\text{ø} = tan^2(45 + \text{ø}/2)$$

$\text{ø} =$ angle of internal friction for the rock mass

Bearing capacity factors (Bell solution)

Bell solution is used for bearing capacity calculation in weak rock. This analysis is used for the rock in the active wedge and the confinement provided by the surrounding rock [14]. The relation used for the Bell solution is:

$$q_a = C_{f1}cN_c + C_{f2}(B\gamma_r/2)N_\gamma + \gamma D N_q\ /\ FS$$

Where B= Footing width

c = Cohesion of rock mass

$D =$ Depth of the foundation below the ground surface

$\gamma_r =$ Density of the rock

$$N_c = 2N_\text{ø}^{1/2}(N_\text{ø} + 1)$$

$$N_\gamma = 0.5N_\text{ø}^{1/2}(N_\text{ø}^2 - 1)$$

$$N_q = N_\text{ø}^2$$

$$N_\text{ø} = tan^2(45 + \text{ø}/2)$$

$B =$ Width of the foundation

FS = Factor of safety

C_{f1}, C_{f2} are the correction factors

Compressive failure

This case characterized for open joints, illustrated in Fig. 2. The failure mode in this case is similar to unconfined compression failure [13].

The ultimate bearing capacity estimated by this Equation:

$$q_{ult} = 2c\ tan\ (45 + \text{ø}/2)$$

$c =$ cohesion

$q_{ult} =$ the ultimate bearing capacity

$\text{ø} =$ Angle of internal friction

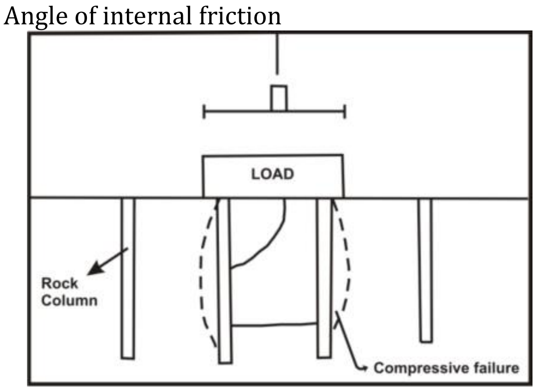

Fig. 2. Compressive failure of open joints *(Modified after U.S.A.C [13])*

RESULTS AND DISCUSSION

Three boreholes (BH-1, BH-2 and BH-3) were drilled on the abutments and river bed to get the subsurface geotechnical information (Fig. 3). Lithological logs and other parameters like core recovery and rock quality designation were measured from these drilled cores. All boreholes were divided into five zones on the basis of variation in lithology, strength and RQD from lithological logs.

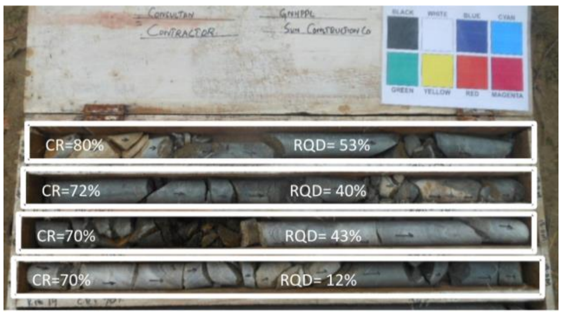

Fig. 3. Rock core box with labeled Rock Quality Designation (RQD) and Core Recovery (CR)

Different parameters for RMR and GSI calculation of each zone were derived by these lithological logs. Detail description of borehole zones with depth, calculated RMR with values & class number and GSI with values & its description are given in (Table 1).

According to the RMR rating quality of the rock is poor and fall in class IV, RMR values varies from 21-32 in BH-1, 22-34 in BH-2 and 21-37 in BH-3. Similarly GSI values varies from 39-48 in BH-1, 44-58 in BH-2 and 44-58 in BH-3, which shows the quality of the rock is blockyto very blocky. Comparision of RMR and GSI (Fig. 4) shows the fluctuation of RMR and GSI in every subsurface zone of borehole.

Table 1. Rock Mass Rating (RMR) and Geological Strength Index (GSI) of BH-1, BH-2 and BH-3

BH-No.	Depth	Zones	RMR			GSI Values
			Values	Class Number	Description	
BH-1 (Right Abutment)	8.5-10.5	1	32	iv	Poor rock	44
	10.5-11	2	24	iv	Poor rock	39
	11-14	3	24	iv	Poor rock	44
	14-19.5	4	21	iv	Poor rock	46
	19.5-20	5	32	iv	Poor rock	48
BH-2 (River Bed)	7-9.5	1	27	iv	Poor rock	49
	9.5-12	2	29	iv	Poor rock	48
	12-14.5	3	34	iv	Poor rock	58
	14.5-20	4	22	iv	Poor rock	44
	20-25	5	27	iv	Poor rock	49
BH-3 (Right Abutment)	3.3-6	1	21	iv	Poor rock	44
	6-9	2	33	iv	Poor rock	49
	9-14	3	25	iv	Poor rock	55
	14-21	4	33	iv	Poor rock	55
	21-25	5	37	iv	Poor rock	58

The rock mass parameters used for the calculation of bearing capacity are cohesion (c), friction angle (ø), density (γ). RocLab (Rocscience, version 1.032) computer program is used for the determination of cohesion (c) and friction angle (ø) of rock mass.

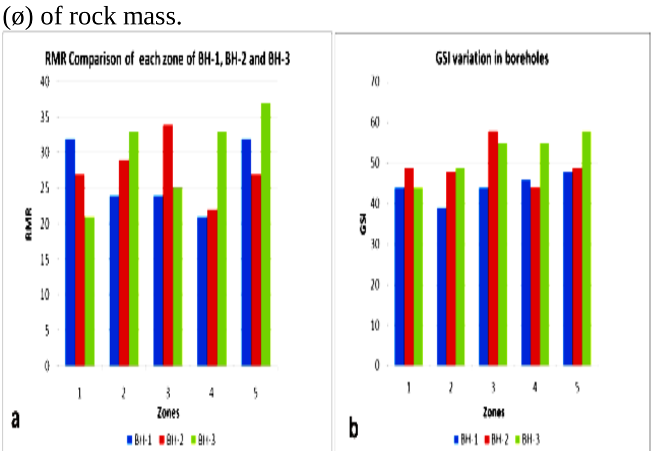

Fig. 4. a) Variation of RMR values in borehole zones, b) Variation of GSI values in borehole zones

The values of cohesion (c) vary from 1.50-2.82 in BH-1, 1.46-3.04 in BH-2 and 1.33-2.82 in BH-3, friction angle (ø) vary from 35-41 in BH-1, 38-41 in BH-2 and 37-43 in BH-3 and density of the rock is 0.027 MN/m^3 is same for all boreholes. Foundation width is 15m and depth of the foundation is varying according to the depth of borehole zones, which are used to calculate the bearing capacity factor by Terzaghi's equation. Bearing capacity of each zone of borehole is calculated for the assessment of various depths for weir foundation and calculated allowable of bearing capacity of boreholes are given in Table 2.

The range of calculated allowable bearing capacity by Terzaghi's equation is 16.30Mpa-26.81Mpa in BH-1, 17.4Mpa-29.93Mpa in BH-2 and 12.15Mpa-24.93Mpa. The range of calculated allowable bearing capacity by Bell Solution is 16.97Mpa-27.84Mpa in BH-1, 17.96Mpa-31.14Mpa in BH-2 and 12.58Mpa-25.96Mpa. The range of calculated allowable bearing capacity by compressive failure equation is 2.22Mpa-3.99Mpa in BH-1, 2.16Mpa-4.44Mpa in BH-2 and 1.85Mpa-3.99Mpa. Average bearing capacity of each zone of all three boreholes is calculated for comparison of these three different equations (Table 2). By Terzaghi's equation maximum average bearing capacity 25.71Mpa and minimum average bearing capacity is 17.82. By Bell's equation maximum average bearing capacity 26.76Mpa and minimum average bearing capacity is 18.53. By Compressive failure equation maximum average bearing capacity 4.03Mpa and minimum average bearing capacity is 2.31Mpa.

The gradual change of bearing capacity in five zones of all boreholes is given in Fig. 5 that shows similarity of increase and decrease in values of each equation. In zone-1 bearing capacity by all three equation shows higher value after this in zone-2 all three equations shows decrease in values in zone-3 values are again increased, in zone-4 values are decreased and in zone-5 values are increased. Fig. 5 shows that all equations have same trend of increase or decrease in bearing capacity values.

Table 2. Average allowable bearing capacity of boreholes

Zones	Allowable bearing capacity by Terzaghi equation (BH-1, 2 &3)				Allowable bearing capacity by Bell's equation (BH-1, 2 &3)				Allowable bearing capacity by Compressive failure equation (BH-1, 2 &3)			
	BH-1 (MPa)	BH-2 (MPa)	BH-3 (MPa)	Average (MPa)	BH-1 (MPa)	BH-2 (MPa)	BH-3 (MPa)	Average (MPa)	BH-1 (MPa)	BH-2 (MPa)	BH-3 (MPa)	Average (MPa)
1	21.89	27.96	20.39	23.41	22.79	29.13	21.27	24.40	3.67	4.35	3.6	3.87
2	16.30	25.02	12.15	17.82	16.97	26.05	12.58	18.53	2.99	3.99	1.85	2.94
3	22.29	29.93	24.93	25.71	23.20	31.14	25.96	26.76	3.67	4.44	3.99	4.03
4	17.86	17.41	22.37	19.22	18.44	17.96	23.08	19.83	2.22	2.16	2.54	2.31
5	26.81	23.02	21.79	23.87	27.84	23.90	22.62	24.79	3.99	3.6	3.45	3.68

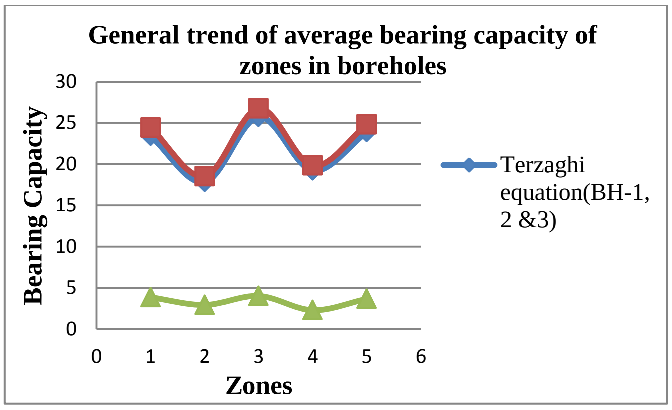

Fig. 5. General trend of bearing capacity in Boreholes

Correlation of RMR and bearing capacity shows the trend of change in calculated bearing capacity according to RMR in boreholes zones (Fig. 6). In correlation of RMR and bearing capacity equations shows that both RMR and allowable bearing capacity is changed simultaneously. If value of RMR is increasing in a zone, similarly allowable bearing capacity values are also increasing. Similarly where RMR is decreasing, allowable bearing capacity values are also decreasing.

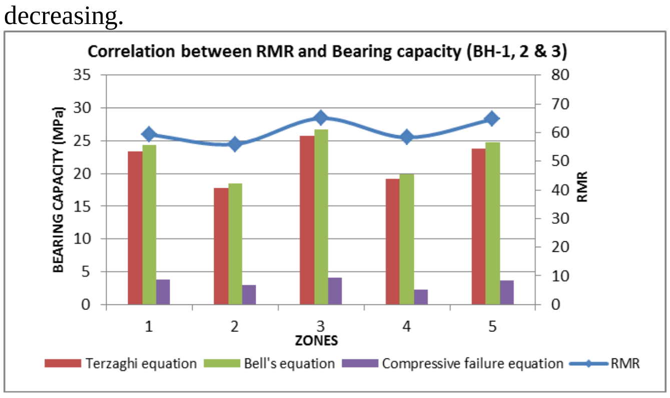

Fig. 6. Correlation between Rock Mass Rating (RMR) and bearing capacity of boreholes

CONCLUSIONS

RMR and GSI are used to classify the rock mass and allowable bearing capacity is calculated for the assessment of various depths of weir foundation. The range of calculated allowable bearing capacity by Terzaghi's equation is 17.82-25.71 MPa and by Bell's equation is 18.53-26.76 MPa. Calculated bearing capacity values lie on the higher side. The allowable bearing capacity assessed by compressive failure equation (2.31-4.03 MPa) is reasonable according to the open joints conditions of the study area. RMR and bearing capacity trend suggests that foundation may be placed on the upper layer of rock but the allowable bearing pressure will require removal of top 0.5-1m rock cover to obtain relatively fresh rock. The weir can safely be abutted to the bed rock on both left and right abutments.

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