



Engineering geological characterization for evaluation of seepage and geomechanical properties of Chalchal Dam Site, Bale Zone, Southeastern Ethiopia

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Abstract

Ethiopia is currently developing irrigation schemes like micro dams to improve its food security. Chalchal Dam project is one of such dams to be constructed in the Bale Zone of Southeastern Ethiopia. However, this dam is subjected to engineering geological problems due to the presence of carbonate and shale rocks in this area. To address this problem, engineering geological mapping, discontinuity surveying, core drilling, geophysical surveying, in situ permeability, and laboratory testing were carried out. From these integrated investigations, the dam site is underlain by colluvial and alluvial soil deposits, sandstone, limestone with decomposed shale, shale, marly limestone, and massive limestone rocks with different degrees of weathering and fracturing. The Lugeon test conducted along the dam axis revealed the presence of potential seepage zones at the left abutment and river center in which the former is underlain by highly permeable rocks with Lugeon values as high as 39.58 and the latter with permeable rocks with a Lugeon value of 9. The geo-mechanical properties of rocks such as friction angle, cohesion, and modulus of deformation at the dam site, as determined using Hoek–Brown failure criteria, were in ranges of 21.52 to 32.06°, 0.123 to 3.006 MPa, and 0.747 to 9.308 GPa, respectively. The bearing capacity of the dam foundation was determined using empirical methods and the result pointed out that the allowable bearing capacity of the dam foundation ranges from 0.116 to 5.321 MPa. After the removal of colluvial and alluvial deposits along the dam axis, ground improvements were recommended along the dam axis mainly in terms of curtain grout and slurry trench or concrete cutoff.

Keywords Lugeon Test · Seepage · Geo-mechanical Properties · Ground improvement · Chalchal Dam · Ethiopia

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Introduction

One of the major challenges for economic development in developing countries is attaining food security. In sub-Saharan Africa countries and Eastern Africa in particular, variation in rainfall together with extended duration of drought period is threatening food and water security (De Hamer et al. 2008). GWP (2010) stated that water plays a crucial role in assuring food security, economic growth, and poverty reduction. Water impoundment structures such as Micro-Dam reservoirs (MDRs) play a major role in addressing food security by enhancing agricultural activity through various scales of irrigation schemes (Garrido et al. 2006; Scanlon et al. 2006).

In Ethiopia, the agricultural sector accounts for 80% of employment (World Bank 2008) and contributes to nearly half (47%) of the country's GDP in 2007 (von Braun and Olofinbiyi 2007). However, more than 80% of Ethiopia's population relies on a rain-fed agriculture which is affected

by uneven and erratic distribution of rainfall. This is even more severe in arid and semi-arid parts of the country. These phenomena have caused a reduction in the productivity of agriculture and frequent drought in the country. However, the country owns about 12 major drainage basins (MoWR 2001) with around 3.7 million ha of nearby irrigable land (WSDP 2002). Of these potentially irrigable lands, only 4.3% was used (FAO 2005) indicating irrigation production of the country is far from pleasing. In recent years, however, the government of Ethiopia is implementing the construction of water harvesting structures like micro dams on some of these drainage basins for the development of irrigation to improve food security in which the Chalchal micro dam is one of them.

Worldwide, the construction of dams has experienced different types of engineering geological problems such as leakage, uplift pressure, differential settlement, abutment instability etc. Leakage is one of the most common problems (Mozafari et al. 2011) particularly in embankment dams where it contributes to nearly half its failures all over the world (Foster et al. 2000). Particularly, those dams constructed on carbonate rocks have suffered excessive leakage attributing to their soluble nature and karstification (Morteza 2012). Hence, many researchers have devoted their works on investigating such problems (Malkawi and Al-Sheriadeh 2000; Nusier et al. 2002; Romanov et al. 2003; Ghobadi et al. 2005). Nevertheless, in spite of numerous ground improvement techniques forwarded by different researchers, leakage is still one of the major problems particularly for those dams founded on sedimentary terrain (Kamal 2007).

To ensure the safety of the dam against such problems, engineering geological investigation should be carried out prior to the construction of these micro dam structures (Sisakian et al. 2019) mainly through discontinuity surveying, core drilling, and in situ testing (Özsan and Akin 2002). The hydraulic conductivity and geo-mechanical properties of rock masses are the major parameters that are investigated through such techniques (Gurocak and Alemdag 2011). Water pressure or the Lugeon test is the most widely deployed technique in acquiring rock mass hydraulic conductivity (Quinones-Rozo 2010; Ghafoori et al. 2011) and can be used to delimit zones of the dam that exhibit different rock mass quality (Foyo et al. 2005). In addition, together with a degree of jointing of rock mass in drill cores, this serves as a criterion in determining the necessary ground improvement (Foyo et al. 2005; Sharghi et al. 2010). Application of grouting in improving the strength of zone of weakness and limiting leakage in dams are quite common (Wahlstrom 1974; Houlsby 1990). Comprehensive descriptions on the application of grouting regarding its type, depth, orientation, type of stage, and the required pressure in dam construction are given by several researchers (Wahlstrom

1974; Houlsby 1990; Ewert 1997; Stare et al. 2013; Weaver 1991; Warner 2004; Fell et al. 2015).

In Ethiopian context, construction of micro dams particularly for irrigation purposes began in the late seventies to combat drought and increase food security (Tiruneh 2005), and they continued to be constructed at a rapid rate in recent years (Berhane, 2010). Studies by several researchers have shown that some of these micro dams have failed while most of them are facing a multitude of geotechnical problems. For instance, most of the micro dams constructed in the northern part of the country particularly on sedimentary terrain of Mekelle Outlier are not serving as planned mainly due to leakage, sedimentation, and hydrological problems (Desta 2005; Haregeweyn et al. 2006; Abdulkadir 2009; Berhane, 2010; Berhane et al. 2013; Abay and Meisina 2015). For example, Haregeweyn et al. (2006) and Berhane et al. (2013) stated that more than half of the dams constructed in this part of the country have experienced excessive leakage problems. Therefore, particular attention and consideration should be given for these problems in planning and designing of dams that are intended to be constructed in related geological setting.

This research work deals with an engineering geological investigation of the Chalchal dam site, which is located in the sedimentary terrain of the Ogaden Basin with the main objectives of determining the hydraulic conductivity and geo-mechanical properties of rock masses. The proposed dam will have a height of 47 m and a dam axis length of 685 m. Engineering geological mapping, discontinuity surveying, core drilling, permeability testing and sampling, and laboratory testing of samples were mainly carried out in order to come up with a comprehensive result which can help in recommending proper site-specific remedial measures.

Location of the study area

The Chalchal dam site is located near the border between Ginir and Raytu Districts of Bale Zone in Oromia Regional State, Southeastern Ethiopia. It is situated in the western part of the Ogaden Basin which is around 630 km from Addis Ababa, the capital of Ethiopia. More precisely, it is bounded between 764,500 and 767,500 m N and between 721,600 and 725,600 m E in the UTM Zone of 37 N (Figs. 1 and 2).

Methodology

In this study, engineering geological mapping, discontinuity surveying, borehole logging, in situ permeability testing, sampling, and laboratory testing of rocks and soils were conducted.

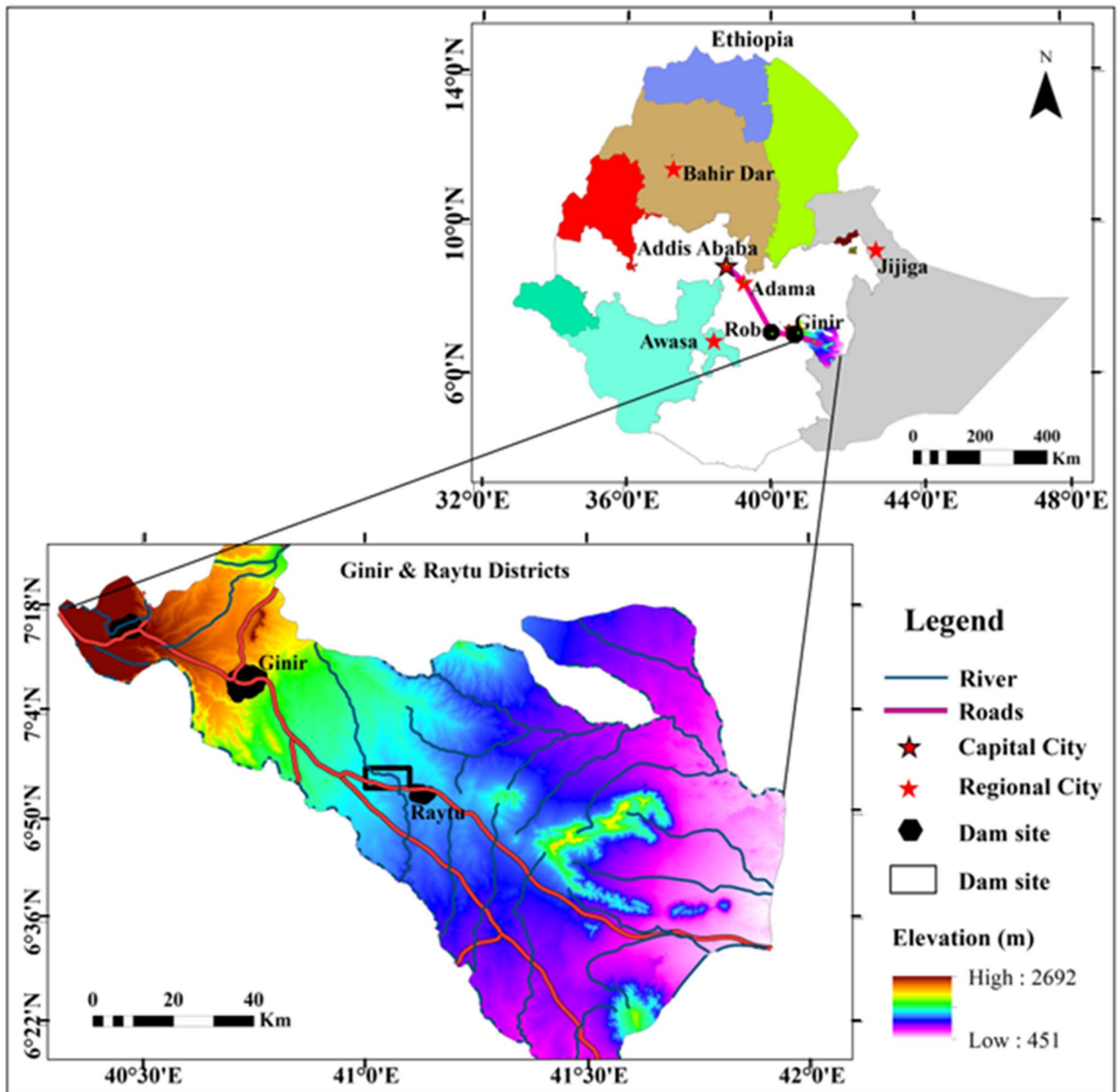


Fig. 1 Location map of the study area and its surrounding

Engineering geological mapping

Engineering geological mapping was carried out along the NE-SW trending traverse line to intersect the major geological units and structures in the area. The descriptions of rock units were performed mainly in terms of the degree of weathering, fracturing, and their response to blow by a geological hammer. Besides, sampling and laboratory testing of rocks and soils were also carried out. Finally, an engineering geological map of the dam site was prepared

by integrating the data obtained from field descriptions and laboratory testing of rocks and soils.

Discontinuity survey

A detailed discontinuity survey was carried out with more emphasis being given to the steeper parts of the dam site. Accordingly, the characteristics of discontinuities such as the orientation, spacing, persistence, aperture, roughness, infill material, wall weathering, and associated

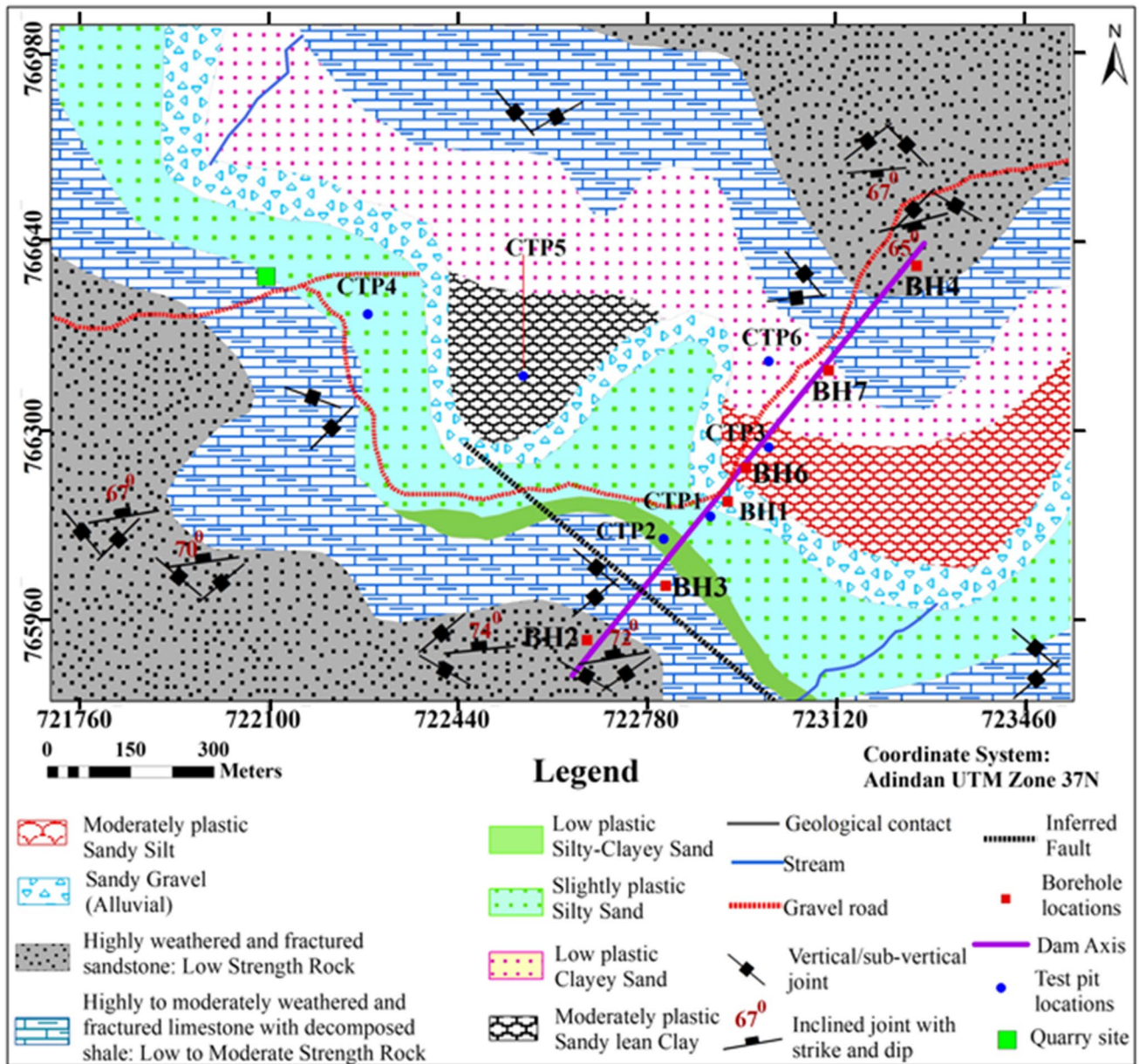


Fig. 2 Engineering geological map of the Chalchal dam site

groundwater conditions were measured and described as per ISRM (1981) guideline. The results of this survey were then used to provide the basic parameters for rock mass classification. Moreover, the orientation data obtained from this survey was also analyzed using dips 6.0 software (Rocscience 2004a, 2004b) in the form of a Rosset diagram which helps to examine the nature of joints and their influence on the dam to be constructed.

Geophysical survey, core drilling, and permeability tests

A geophysical survey was conducted using vertical electrical sounding (VES) to investigate the subsurface condition of the dam site. VES data at each survey location was processed using Ipi2win software and later converted into a geo-electric section using Surfer 16 software. Interpretation

of VES was made by considering the geology of the area and was later verified through correlation with drilling results.

Then, a geotechnical drilling was conducted with 7 boreholes being drilled out of which 6 of them were located along the proposed foundation of the dam to obtain subsurface information through core logging, in situ permeability tests, and recovery of core samples for laboratory testing. The boreholes were drilled to a depth range of 10 to 40.25 m with a total core length of 174.5 m. Core logging and associated description and measurement of the discontinuity characteristics and RQD were carried out in accordance with ISRM (1981) recommendations.

In situ permeability tests conducted upon completion of the drilling operation include falling head and Lugeon tests. The former was conducted for relatively soft topmost geological formations while the latter was utilized for relatively sound formations of the dam site. The Lugeon test was conducted by setting a test section length up to 5 m and by applying test pressures which have an increasing and then decreasing trend in a total of five stages. The test pressure was usually decided based on column and formation pressures. Then, the amount of water flows into the test section per minute with a corresponding pressure and Lugeon value were determined. Later, interpretations of Lugeon values were made based on the flow pattern of Houlsby (1976). Moreover, the rock mass permeability of the dam site was classified using Lashkaripour and Ghafoori (2002) and Canoğlu et al. (2017) methods.

Results and discussions

Site geology

Geological factors are the most important natural factors that influence the design of dams (Lashkaripour and Ghafoori 2002). In this study, a geological investigation was carried out to obtain preliminary information regarding the influence of site geology on the dam to be constructed.

The major rock units that are outcropped within the dam site are Mesozoic sedimentary rocks such as limestone-shale intercalation, sandstone, and Quaternary alluvial and colluvial deposits (Fig. 2). The limestone-shale intercalation is

horizontally bedded and mainly outcropped along the gently sloping part of the abutments of the dam's upstream and downstream sections. The limestone beds in this intercalation unit are highly to moderately weathered and fractured. The layer of shale within this intercalation is weak, fissile in places, and extremely weathered to totally decompose into a fine soil. Hence, it can introduce groutability problems owing to its decomposition. The sandstone unit overlies the intercalation and is exposed forming cliffs. It is weak, highly friable, and extremely weathered rock which can easily disintegrate into pieces by a single hammer blow. It is also highly fractured with closely to widely spaced, highly persistent, and open to partially filled vertical to steeply dipping systematic and non-systematic joints. Colluvial deposit is found overlying the limestone-shale intercalation in most localities while the alluvial deposits are limited to the center and banks of the river.

The colluvial soils are categorized into slightly to low plastic silty sand (SM), clayey sand (SC), and silty clayey sand (SM-SC) with a plasticity index of 3.64%, 8.26%, and 6.72%, respectively. The first two are characterized by their semi-pervious nature with a coefficient of permeability in order of 1.301×10^{-5} and 2.526×10^{-6} cm/s respectively. Alluvial deposits at the banks of the river mainly comprise medium plastic sandy silt (ML) and sandy lean clay (CL). At the river center, however, this deposit consists of pervious sandy gravel with a coefficient of permeability in the order of 3.290×10^{-4} cm/s. Hence, it is proposed to remove both colluvial and alluvial deposits from the foundation of the dam axis to prevent excessive leakage, uplift pressure, settlement, and their erosion through migration into open joints.

Discontinuity data

A detailed discontinuity survey was carried out in both abutments of the dam to acquire data on discontinuity parameters. Table 1 and Fig. 3 show characteristics of major discontinuity sets and Rosset diagram of discontinuity measurements obtained from left and right abutments. These measurements were taken systematically on those joints with favorable orientation to seepage. The discontinuity sets were distinguished visually based on the Rosset diagram and field observation. Accordingly, three dominant joint sets, namely,

Table 1 Characteristics of major discontinuity sets

Discontinuity sets	Average spacing (m)	Average aperture (m)	Average persistence (m)	Roughness	Degree of weathering
NW–SE (JS1)	0.462	0.0132	22.5	Slightly rough	Decomposed to moderately weathered
NE–SW (JS2)	0.365	0.0108	15.2	Moderately rough	Slightly to highly weathered
ENE–WSW (J3)	0.423	0.0093	29.6	Slightly rough	Slightly to moderately weathered
Horizontal (J4)	0.241	0.0034	30.2	Moderately rough	Moderately to highly weathered

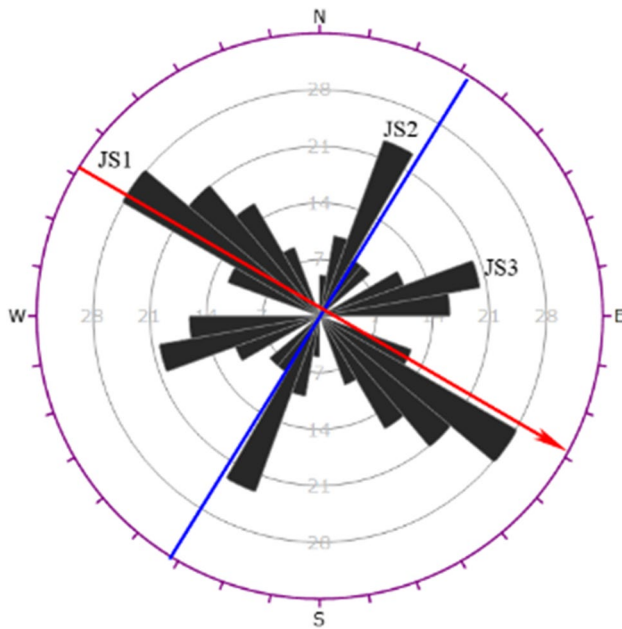


Fig. 3 Rosset diagram of discontinuity strikes plotted by dip.6.0 software (Blue and red lines represent the approximate alignment of the dam axis and river flow direction respectively)

NW–SE (JS1), NE–SW (JS2), and ENE–WSW (J3) were identified (Fig. 3). These joint sets are more or less oriented similar to the regional geological structures. In addition, the bedding plane, J4, is horizontal. As shown in Fig. 3, NW–SE trending JS1 and the river flow direction (which is indicated by the red arrow) are oriented nearly parallel to each other. In the dam site, joints that are parallel the river flow direction favor seepage (Maerz and Zhou 1999; Abay and Meisina 2015). Thus, JS1 has a favorable orientation for seepage in this dam site. In general, discontinuities of this dam site are closely to widely spaced and highly persistent in the exposed surfaces and beyond. The average spacing and aperture range from 0.241 to 0.462 m and from 0.0034 to 0.0132 m, respectively.

Borehole logs

Proper logging of geological formation of the dam site shows a complex sedimentary succession. The studied core samples show variation in the nature of strata, degree of weathering and fracturing, and their overall geotechnical properties. The geology of left and right abutment of the dam is similar and underlain from top to bottom by colluvial deposit and variably fractured and weathered sandstone, limestone with decomposed shale, shale, marly limestone, limestone with decomposed shale, and massive limestone rock units (Figs. 4a, c; 5a; and 6). The geological sequence of the river center is similar to those of the left and right abutment of the dam except that there are alluvial deposits (sand-gravel

mixture and sandy silt) with the thickness of 18 m covering the center and left of bank of the river (Figs. 4b, 5b, and 6).

Geophysical investigation

Geophysical investigation was also carried out using vertical electrical sounding (VES) before geotechnical drilling. This VES survey was conducted along the NE–SW profile direction in the dam axis at 6 VES locations (Fig. 7). The VES data at each location was interpreted individually in terms of layer thickness and resistivity. Later, interpreted VES curves were combined to produce the geo-electric section of the dam site. The interpretation was made based on the geology of the area, and later, it was correlated with the borehole data. Consequently, about five geo-electric layers were identified from the combined interpreted VES curves. The first layer is the top soil which is a mixture of clay, silt, sand and gravel with an apparent resistivity ranging from 6 to 110 $\Omega\cdot\text{m}$ (Fig. 7). According to Daniels and Alberty (1996), this variation in resistivity values may be attributed to moisture content and grain size properties of soils. The second layer with resistivity ranging from 4 to 50 $\Omega\cdot\text{m}$ is interpreted as alluvial sediment and is limited to the river course. Core drilling results at BH1 and BH7 also revealed the presence of this layer. The third geo-electric layer has an apparent resistivity ranging from 8 to 63 $\Omega\cdot\text{m}$ (Fig. 7). Vandecasteele et al. (2011) stated that discontinuities and weathered horizons produce low resistivity anomaly zones. Hence, this layer was interpreted as a highly weathered and fractured rock formation, and it can be a potential seepage zone. From the core drilling result of boreholes located along the dam axis, this layer corresponds to sandstone, limestone-shale intercalation, shale, and marly limestone rocks. Therefore, the VES survey failed to distinguish these low resistivity geological formations. The bottom layer of this geo-electric section is characterized by relatively high apparent resistivity in the range of 1000–3000 $\Omega\cdot\text{m}$ and was interpreted as massive limestone.

Interpretation of VES data through the construction of geo-electric section also confirmed the presence of a weak zone (fault) at the right abutment of the dam which was inferred during the field geological mapping. Besides, this geo-electric section also depicted the presence of another fault passing through the left abutment of the dam as well (Fig. 7). A fault within the dam site, among many factors, leads to water-tightness, differential settlement, and seismicity problems (Tosun 2011). Therefore, these faults need a very detailed investigation before the dam is constructed.

RQD analysis with depth

RQD has been used as the first rock mass quality assessment parameter with depth in the design of many dams (Ghazifard

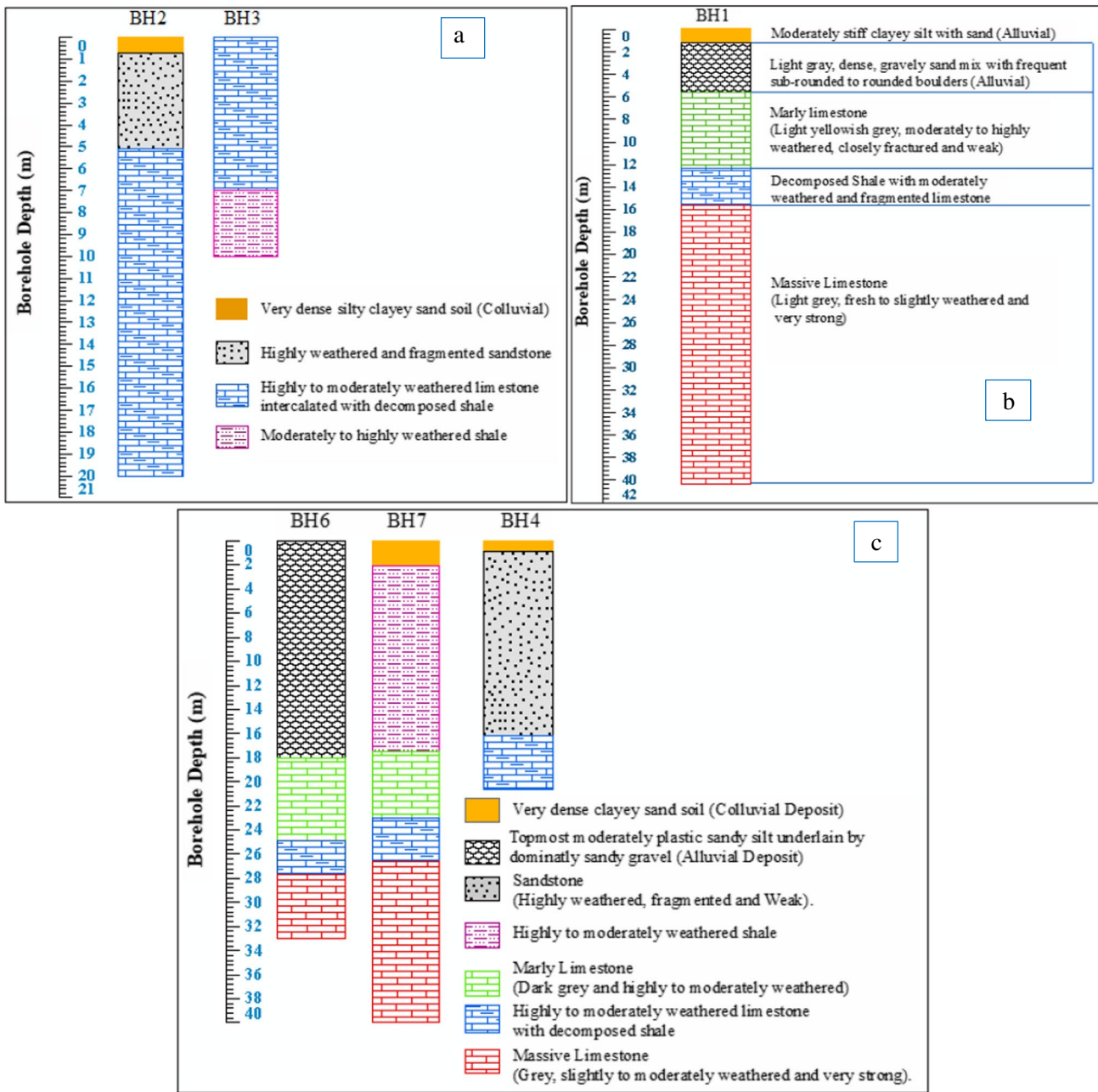


Fig. 4 Lithological log of the boreholes: **a** BH2 and BH3 at the right abutment, **b** BH1 at river center, and **c** BH4, BH6, and BH7 at the left abutment of the dam

et al. 2006). It provides useful information on the distribution of rock types, degree, and depth of rock weathering and fracturing (Deere and Deere 1988). In this view, the analysis of RQD with depth was made for 5 boreholes that were drilled along the dam axis from their corresponding borehole log data. Moreover, the statistical distribution of RQD values in different rock quality classes for different rocks of the dam site was also analyzed.

Figure 8 illustrates the plot of RQD with depth for 5 boreholes drilled at the river center, right and left abutments of

the dam. The analysis showed that RQD records were plotted for very poor to fair RQD classes up to the depth intervals of 19.05–20.65 m, 28.5–30.0 m, and 36.0–37.0 m below BH1, BH6, and BH7, respectively (Fig. 8a, d, e). Beyond these depth intervals, almost all RQD values fall in a good to excellent rock mass quality classes in these boreholes. From the geotechnical cross-section shown in Fig. 6, the depths of low RQD correspond to the elevation of 1090.5 m, 1088 m, and 1089 m below BH1, BH6, and BH7, respectively. On the contrary, majority of the calculated RQD records were

Fig. 5 Selected core samples from Chalchal dam site: **a** sandstone rock unit (BH4, depth interval: 1.00–6.00 m), **b** massive limestone rock unit (BH1, depth interval: 17.85–21.05 m)

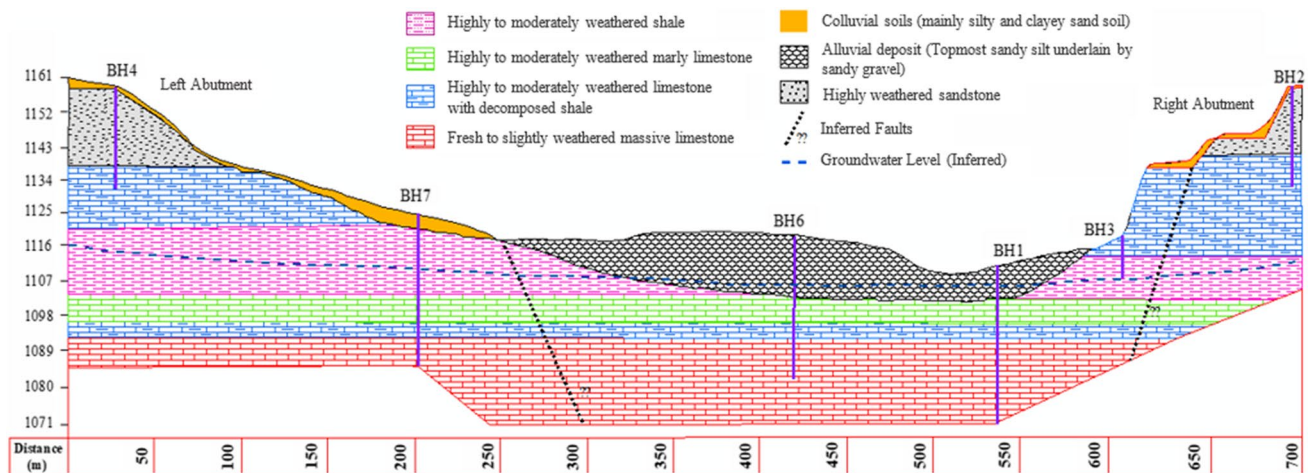


Fig. 6 Engineering geological cross-section from six boreholes along the dam axis (facing downstream)

plotted in very poor to poor rock mass quality classes for the entire drilled depth for BH4 and BH2 (Fig. 8b, c).

Although depth ranges in the low RQD rocks have been differentiated, the plot of RQD values did not show any clear trend with depth (Fig. 8 a, c, d, and e). This is possibly attributed to changes in rock type and the degrees of fracturing with depth. In such conditions, it is difficult to rely on RQD values for deciding on the groutability of materials and selecting depth sections for permeability tests (Uromeihy and Farrokhi 2011; Berhane and Walraevens 2013). Therefore, for this dam, RQD should be integrated with other techniques such as water pressure for deciding the groutability

of rocks below the dam foundation. Moreover, RQD should not also be used as a criterion for deciding test sections for permeability tests in the detailed design and construction phases.

The basic statistical distributions of RQD values show variation in the RQD of rocks at different sections of the dam. Accordingly, 75% of the determined RQD values of a sandstone rock unit fall in the very poor/poor quality class (Table 2). Similarly, all the entire determined RQD values of shale and limestone in the intercalation with shale (Table 2) are also classified in a very poor/poor RQD class in all sections of the dam. The low RQD values of the limestone in

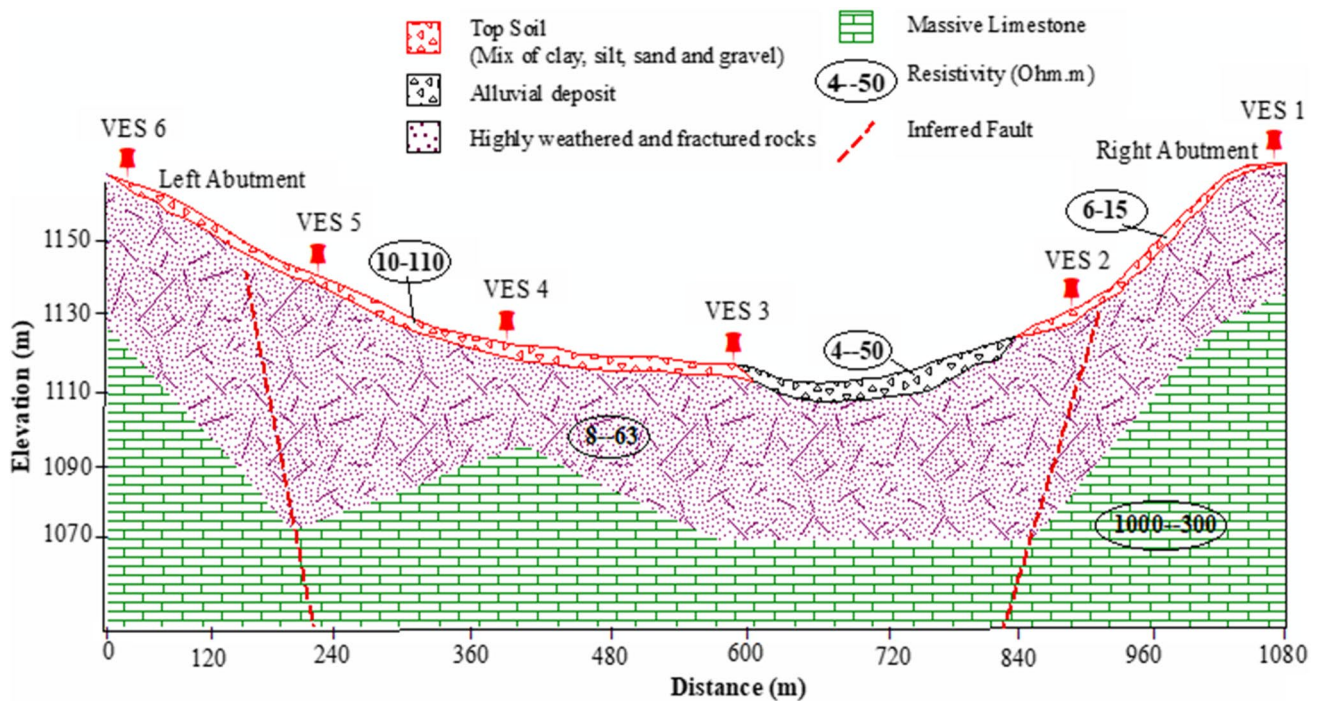


Fig. 7 Geo-electric section along the dam axis (modified after OWWDSE, 2018)

the intercalation unit may be attributed to its high degree of jointing while for the shale unit, it is mainly attributed to its inherent weakness and affinity to lose strength when it is saturated. For marly limestone, no cores of this rock unit greater than 10 cm were recovered within the left abutment of the dam at BH6. In addition, 75% of its RQD at this abutment at BH7 and 66.67% of its RQD at the river center at BH1 were classified into a poor-quality class. This showed that this unit is highly jointed and fragmented and possibly with some solution cavities. On the other hand, massive limestone has better rock quality as high as 70.59% and 75%. Moreover, 28.56% of the RQD values for the massive limestone fall into good to excellent quality classes at BH1, BH6, and BH7 respectively. This showed that the rock is relatively intact, less fractured and altered.

Permeability

The hydraulic conductivity of the rock mass at the dam site was evaluated via Lugeon and falling head permeability tests. The results were interpreted with respect to different rock types of the dam site as shown in Table 3. At the left abutment of the dam, a falling head test was conducted on the topmost sandstone unit, and the test revealed its semi-pervious characteristics with a hydraulic conductivity of 1.15×10^{-5} cm/s. But the underlying limestone-shale intercalation unit was not tested. The hydraulic conductivity of the remaining rock units in the dam site was tested using the

Lugeon test. The shale unit was found to be highly pervious as per Canoğlu et al. (2017) and Lashkaripour and Ghafouri (2002) to the depth of 15 m below BH7 with Lugeon value as high as 38.93 (Table 3). Therefore, this unit is leaky, and potentially high seepage problems are expected in this geologic unit at the abutment. This unusual high Lugeon value of shale is potentially attributed to its high degree of fracturing. In comparison, such a high Lugeon value of shale rock was also common in another sedimentary basin of Ethiopia (e.g., Mekele Outlier) as reported by Abay and Meisina (2015). Below the depth of 15 m in the dam site, the rocks are impervious as the water pressure test yielded a representative Lugeon value of less than 1 (Table 3). According to Houlsby (1990 and 1976) and Uromeihy and Farrokhi (2011), seepage controlling measures such as curtain grout should penetrate at least one stage below the base of a permeable layer. Therefore, at this section of the dam, seepage controlling measures should be provided at least to the depth of 20 m. On the other hand, at the right abutment, hydraulic conductivity was estimated using the falling head test only to the depth of 6 m, and the tested rocks were found to be pervious to semi-pervious as per USBR (1987) rating.

At the river center, a water pressure test revealed a pervious marly limestone unit to a depth of 12.3 m with a representative Lugeon value of 9. The underlying limestone-shale intercalation was not tested to prevent possible hydro-jacking due to its intensive degree of fracturing and weathering, and this is generally considered permeable based on field

Fig. 8 RQD variation with depth **a** at river center, **b** at right abutment, and **c, d, e** at left abutment of the dam

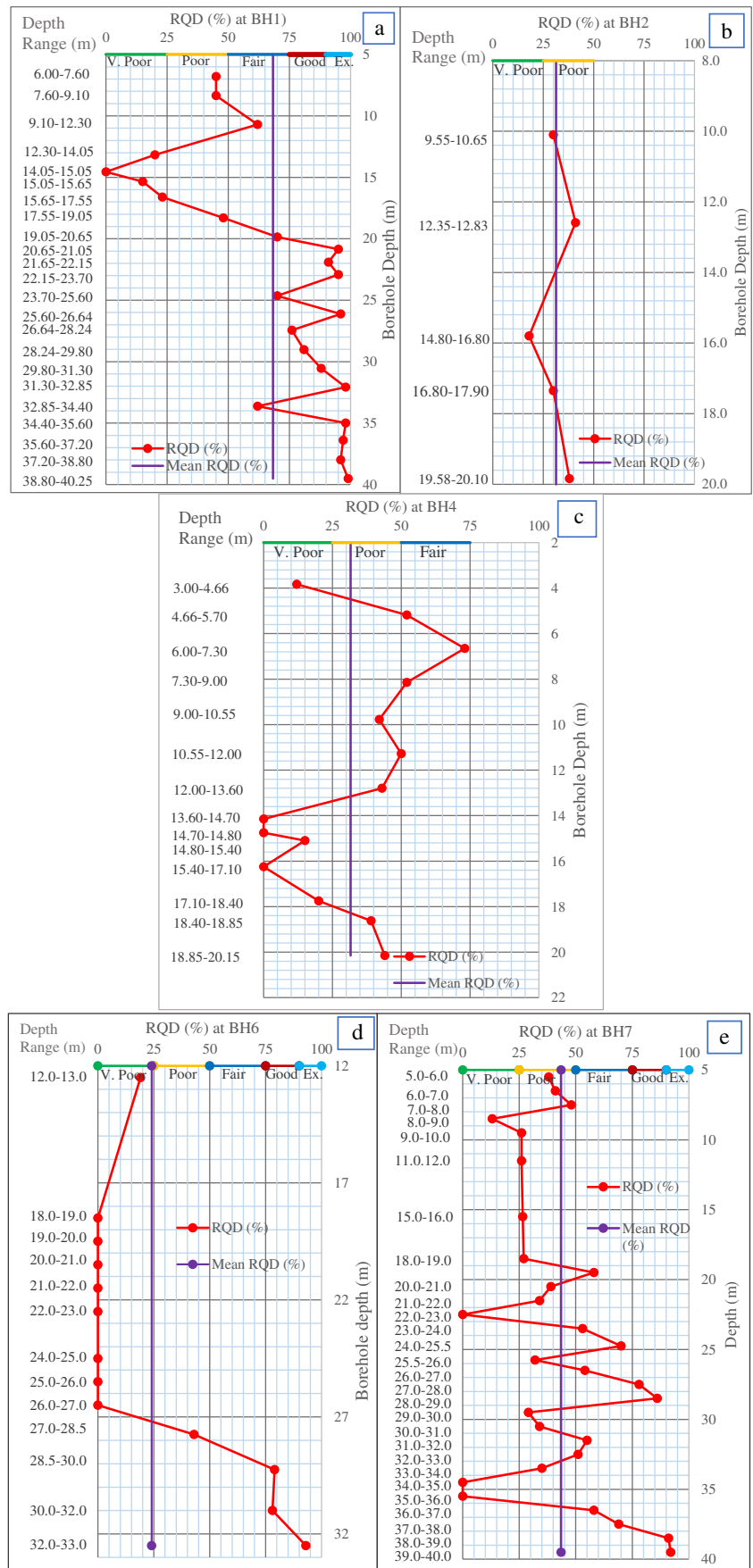


Table 2 Statistical distribution of RQD values of rocks based on Deere and Deere (1988) RQD classification

Location	Rock Type	Statistical distribution of rocks in Deere and Deere (1988) RQD Class					Average RQD(%) and Class	
		RQD Class	Very Poor	Poor	Fair	Good		Excellent
LA (BH4)	Sst	% Frequency in each RQD Class	41.67	33.33	25.00	–	–	31.92/Poor
	Lst		33.33	66.67	–	–	–	34.33/Poor
LA (BH6)	Mlst		100	–	–	–	–	0.00/V. Poor
	Lst		100	–	–	–	–	0.00/V. Poor
LA (BH7)	mLst		–	25.00	–	50.00	25.00	73.25/Fair
	Sh		33.33	66.67	–	–	–	33.16Poor
	Mlst		–	75.00	25.00	–	–	39.50/Poor
	Lst		20.00	40.00	40.00	–	–	38.00/Poor
RC (BH1)	mLst		14.28	21.42	35.71	14.28	14.28	52.28/Fair
	Mlst		–	66.67	33.33	–	–	50.67/Fair
	Lst		100.00	–	–	–	–	11.67/V. Poor
RA (BH2)	mLst		5.88	5.88	17.65	17.65	52.94	81.35/Good
	Lst		20.00	80.00	–	–	–	31.4/Poor

LA left abutment, RC river center, RA right abutment, Sst sandstone, Mlst marly limestone, Sh shale, Lst limestone in the intercalation with shale, mLst massive limestone

Table 3 Results of Packer test and in situ falling head permeability tests (OWWDSE 2018 & 2020)

Packer test results						
Location	Borehole ID	Tested depth (m)	Rock type	Flow behavior	Repr. Lugeon	Permeability class (Canoğlu et al., 2017)
Left Abutment	BH7	5.0–10.0	Sh	Dilation	39.58	Highly permeable
		10.0–15.0	Sh	Washout	38.93	Highly permeable
		15.0–20.4	Mls	Void filling	0.087	Impermeable
		20.4–25.4	Mlst and LstSh	Dilation	0.156	Impermeable
		25.4–30.4	mLst	Dilation	0.069	Impermeable
		30.4–30.5	mLst	Dilation	0.0154	Impermeable
		35.5–40.0	mLst	Void filling	0.0107	Impermeable
	BH6	23.0–28.0	LstSh	Void filling	0.052	Impermeable
		28.0–33.0	mLst	Dilation	0.074	Impermeable
		35.20–40.20	mLst	Dilation	0	Impermeable
River Center	BH1	7.30–12.30	MLst	Dilation	9	Permeable
		19.20–23.70	mLst	Void filling	0	Impermeable
		28.20–33.20	mLst	Dilation	0	Impermeable
		35.20–40.20	mLst	Dilation	0	Impermeable
Falling head permeability test result						
Location	Borehole ID	Test section	Rock type	Hydraulic Conductivity (cm/sec)	USBR (1987) rating	
River center	BH1	2.0–3.0	Alluvial	3.290×10^{-4}	Pervious	
Right abutment	BH2	1.0–3.0	Sst	1.26×10^{-4}	Pervious	
		3.0–6.0	Sst and partly LstSh	1.15×10^{-4}	Pervious	
	BH3	1.0–3.0	LstSh	1.06×10^{-6}	Semi-pervious	
		3.0–6.0	LstSh	9.11×10^{-6}	Semi-pervious	
Left abutment	BH4	1.0–3.0	Sst	1.15×10^{-5}	Semi-pervious	

MLst marly limestone, mLst massive limestone, Sst sandstone, LstSh limestone-shale intercalation, Sh shale

observation. The underlying massive limestone starts from the depth of 19.2 m, and it is found to be impermeable at all test sections (Table 3). Hence, considering assumption that seepage controlling measures should penetrate at least one stage below the base of permeable layer, seepage curtains should be provided for this section of the dam to depth of at least 24 m.

Type of water flow

The result of flow type for the Lugeon test conducted at the river center and left abutment of the dam is plotted as shown in Fig. 9 to examine the characteristics of discontinuity of rock mass. The water pressure tests carried out at the river center and left abutment of the dam predominantly showed dilation and void-filling flow behaviors. Accordingly, 55.56% and 33.33% of flow patterns at the left abutment and 75% and 25% flow patterns at the river center showed dilation and void-filling behaviors, respectively. The

limestone-bearing rocks (i.e., marly limestone, limestone-shale intercalation, and massive limestone) entirely showed these two types of flow behavior. From the geology of the area, it can be noticed that the limestone-bearing rocks are characterized by disconnected discontinuities and perhaps separated solution cavities. The washout behavior was exclusive to shale indicating that this unit is characterized by relatively open discontinuities with a high degree of interconnectedness that allows washing out of the infill materials.

Correlation of Lugeon values with depth and RQD

Correlation of Lugeon values with depth was made for the central foundation and left abutment of the dam. Accordingly, the Lugeon value tends to decrease with depth and approaches zero after a certain depth in both locations (Fig. 10). This decrease in Lugeon value may be attributed to an increase in the overburden effect with depth (Lee and Farmer 1993; Nappi et al. 2005). Besides, it may be related

Fig. 9 Percentage distribution of water flow behaviors at the river center and left abutment

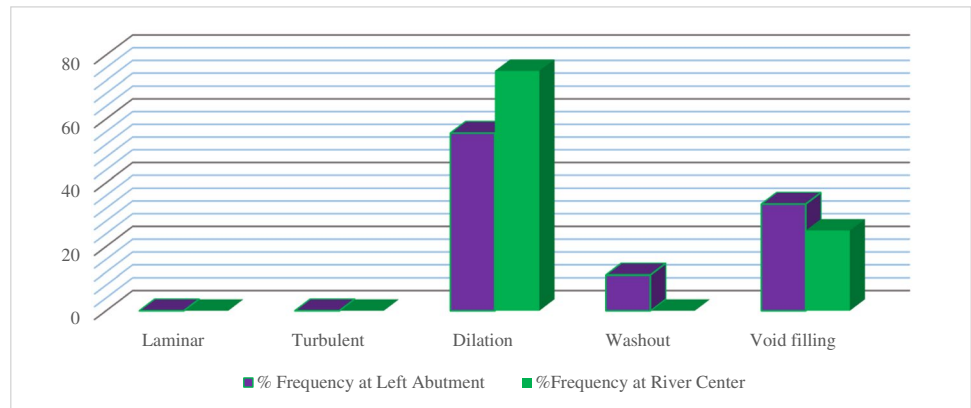
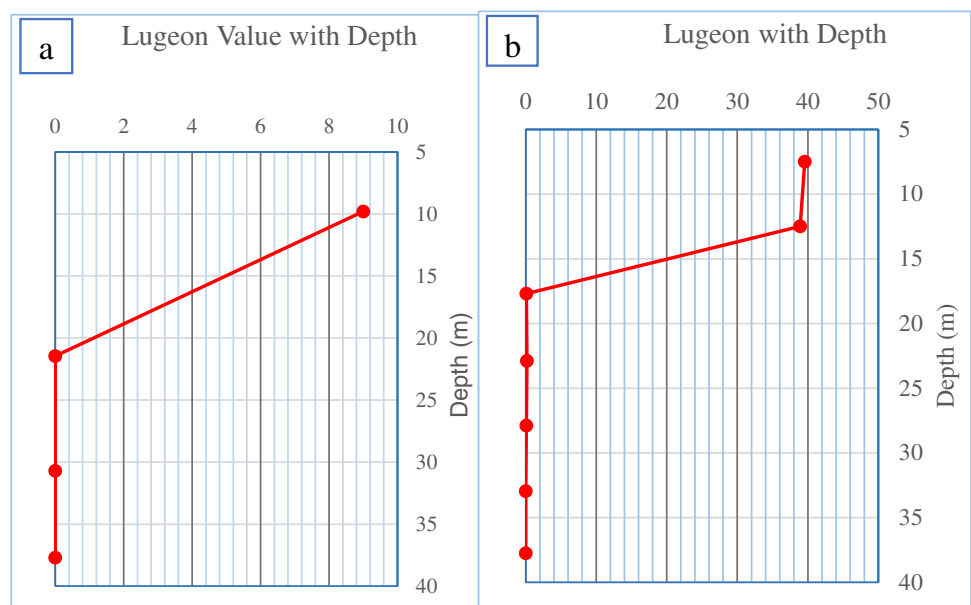


Fig. 10 Variation of Lugeon values with depth for a BH1 and b BH7



to a decrease in the degree of weathering and fracturing with depth.

Similarly, the correlation of Lugeon values with RQD was made for those boreholes for which the packer test was conducted. It can be depicted in most cases that low RQD rocks are highly fractured and hence characterized by high Lugeon values (Reza and Ali 2012). In this study, Lugeon values at the river center tend to show some glimpses of correlation with RQD as highly to moderately weathered and fractured marly limestone of low RQD showed relatively higher Lugeon ($Lu=9$) values while the fresh to slightly weathered massive limestone of high RQD showed low Lugeon ($Lu=0$).

However, several studies have shown that this is not always the case as low RQD rocks can also show low Lugeon values (Foyo et al. 2005; Ghafoori et al. 2011; Berhane and Walraevens 2013). The main reason for such relationship is associated mainly with the failure of RQD to incorporate joint continuity and presence of infill material (Houlsby 1990; Ewert 1997; Choi and Park 2004; Bell 2007; Hamm et al. 2007). From this perspective, Lugeon values at some test sections in the left abutment of the dam tend to be associated with low Lugeon values with low RQD. Such a relationship can be illustrated in Figs. 8e and 10b. For instance, in the depth interval of 5.0–10.0 m at BH7, the shale has got higher average RQD (43%) and higher Lugeon value (39.58) in comparison to a depth interval of 15.0–20.4 m where marly limestone has got lower average RQD (26%) and lower Lugeon value (0.087). Similarly, at an interval of

30.4–35.5 m of this borehole, the massive limestone showed a low average RQD (29%) and a very low Lugeon value (0.0154). Hence, it can be deduced that Lugeon values from the permeability test of boreholes have no direct relationship with the RQD of rocks. Therefore, during detailed subsurface analysis, RQD should not be used as a criterion for deciding the test sections for water pressure tests.

Laboratory tests

The laboratory tests were conducted for intact rocks collected from the boreholes to determine the physicommechanical properties. The test results are listed in Table 4. The laboratory analysis showed that the sandstone and shale rocks are characterized by low UCS with a relatively high water absorption capacity. The combination of these two properties (i.e., low UCS and high water absorption) is the characteristics of weak rocks that make them problematic. The massive limestone, on the other hand, has a relatively higher UCS value and lower water absorption capacity which make them relatively good for dam construction.

Rock mass classification

Rock mass classification systems usually form the pillar of empirical design and are widely utilized in rock engineering (Singh et al. 1999). In this study, the Bieniawski (1989) basic rock mass rating system was utilized to estimate the quality of rock mass constituting the dam site. The input parameters

Table 4 Laboratory test results on intact rock samples from the foundation of the dam axis (OWWDSE 2018 and 2020)

Sample ID	Sample depth (m)	Rock type	WA (%)	UCS (MPa)	Classification of rock from UCS as Per ISRM (1978)
BH1-1	6.0–6.4	Mlst	2.58	23.74	Weak rock
BH1-2	7.6–8.0		3.45	32.85	Medium strong
BH1-3	9.1–9.4		3.23	24.51	Weak rock
BH1-4	18.0–18.5	Lst	1.98	37.040	Medium strong
BH1-5	24.8–25.0		1.72	40.283	Medium strong
BH1-6	27.3–27.7		2.34	48.352	Medium strong
BH1-7	30.1–30.6		2.37	71.409	Strong rock
BH1-8	32.6–32.8		1.34	54.120	Strong rock
BH1-9	34.6–35.0		5.29	35.862	Medium Strong
BH1-10	37.7–38.1		1.54	45.470	Medium Strong
BH4-1	5.7–6.0	Sst	12.16	7.315	Weak rock
BH4-2	6.2–6.4		11.72	6.084	Weak rock
BH4-3	10.5–10.7		12.87	7.834	Weak rock
BH4-4	12.4–12.6		13.04	8.546	Weak rock
BH7-1	11.4–11.8	Sh	3.42	3.23	Very weak rock
BH7-2	12.2–12.7		3.10	3.96	Very weak rock
BH7-3	15.5–15.9		4.62	4.68	Very weak rock

Mlst marly limestone, Lst massive limestone, Sst sandstone, Sh shale

for this analysis include discontinuity parameters, groundwater condition, and RQD which were determined through discontinuity survey of surface outcrop and core log analysis while intact rock strength was determined in the laboratory through point load strength test. This analysis showed that the quality of the sandstone is better at the left abutment than the right abutment of the dam (Table 5). In addition, the analysis also classified the shale unit as poor rock while limestone rock in the limestone-shale intercalation at abutments, marly limestone, and massive limestone at the river center as fair rocks (Table 5).

Estimation of geo-mechanical properties and bearing capacity of rocks

Geo-mechanical properties

In this study, the geo-mechanical properties of a rock mass (i.e., rock-mass compressive strength, cohesion, friction

angle and deformation modulus) of the dam site were determined using Rockdata (Rocscience 2004a, 2004b) software with the Hoek–Brown Failure criteria. The software utilizes uniaxial compressive strength (UCS) of intact rock, geological strength index (GSI), material constant (m_i), and disturbance factor (D) to determine the aforementioned geo-mechanical properties. In this study, UCS of intact rocks that was determined through point load strength test (Table 4) and GSI value, which was determined in the field, were used for the analysis purpose. The material constant (m_i) value for the rocks of the study area was taken from standard table built-in Rockdata software while a disturbance factor of zero was used for the analysis assuming a natural ground condition.

As shown in this Table 6, the friction angle, cohesion, and modulus of deformation of the rocks at the dam site vary from 21.52 to 32.06°, 0.123 to 3.006 MPa, and 0.747 to 9.308 GPa, respectively. This analysis showed that the sandstone and limestone units have a lower friction angle,

Table 5 RMR classification of rocks in the dam site

Input parameters	Left abutment						River center				Right abutment			
	Limestone (in the intercalation)		Sandstone		Shale		Marly limestone		Massive limestone		Limestone (in the intercalation)		Sandstone	
Description	Value	R	Value	R	Value	R	Value	R	Value	R	Value	R	Value	R
Average UCS (MPa)	61.34	7	7.45	2	3.96	1	27.03	4	47.5	4	43.68	4	8.85	2
Average RQD (%)	73.67	13	31.92	8	24.4	3	34.1	8	68.96	13	65.42	13	47.63	8
Joint spacing (cm)	20–60	10	6–20	8	6–20	8	6–20	8	20–60	10	20–60	10	<6	5
Condition of disc		13		12		11		12		19		14		7
Groundwater Cond	Dry	15	Dry	15	Dry	15	Damp	10	Damp	10	Damp	10	Dry	15
RMR value	58		45		38		42		56		51		37	
RMR Class	III		III		II		III		III		III		II	
R	Fair		Fair		Poor		Fair		Fair		Fair		Poor	

Disc. discontinuity, *Cond.* condition, *R* rating

Table 6 Shear strength and deformation parameters of rocks of the dam site based on Hoek–Brown failure criteria

Rock type	Input parameters			Hoek–Brown failure criteria			
	UCS MPa	GSI	m_i	UCS _{rm} (MPa)	C_m (MPa)	ϕ_m (°)	E_m (GPa)
Right abutment							
Sst	8.85	32	17	0.175	0.285	22.40	1.055
Lst	43.68	46	8	2.078	1.901	27.48	5.249
Left abutment							
Sst	7.45	40	17	0.246	0.375	32.06	1.534
Lst	61.34	53	8	4.397	3.006	29.50	9.308
Sh	3.96	33	6	0.084	0.123	21.52	0.747
River center							
MLst	27.03	43	8	1.074	1.118	26.62	3.474
mLst	47.50	51	8	3.032	2.249	28.93	7.300

Sst sandstone, *Lst* limestone in the intercalation, *Sh* shale, *MLst* marly limestone, *mLst* massive limestone

cohesion, and modulus of deformation at the right abutment than the left abutment of the dam.

Bearing capacity

The bearing capacity of the rock mass is usually controlled by condition, spacing, and orientation of discontinuities (Kanik and Ersoy, 2019). It is also strongly controlled by the slope of the ground with respect to the foundation of the engineering work under consideration. It can be determined using analytical, semi-empirical, or in situ methods (Alavi and Sandrossadat 2016), and this study was done based on empirical methods.

For closely fractured and weak rocks that lie on a relatively horizontal surface, Wyllie (1992) has developed the following equation to obtain the allowable bearing capacity (q_a) based on Hoek–Brown Strength Criteria:

$$q_a = \frac{C_{f1} s^{0.5} UCS (1 + ((ms^{-0.5} + 1))}{FOS} \tag{1}$$

where C_{f1} : factor of correction, s and m : rock mass constant, UCS: intact rock strength, and FOS: factor of safety.

On the other hand, Bell (2007) as cited in Wyllie (1992) has forwarded the following equation for calculation of allowable bearing capacity (q_a) of rocks resting on the sloping parts of the foundation:

$$q_a = \frac{C_{f1} c N_{cq} + C_{f2} \left(\frac{B\gamma}{2}\right) N_{\gamma q}}{FOS} \tag{2}$$

where C_{f1} and C_{f2} are correction factors N_{cq} and $N_{\gamma q}$ are bearing capacity factors and are dependent on the slope

angle (β), C is cohesion, B is width of the foundation, FOS is the factor of safety, and γ is the unit weight of the rock.

Accordingly, for weak and closely fractured marly limestone and shale units that lie on a relatively horizontal part of the dam, the allowable bearing capacity was determined using (Eq. 1), and the results are presented in Table 7. As shown in this table, the shale unit has a very low allowable bearing capacity in an order of 0.19 MPa. On the other hand, for sandstone and limestone units that lie on the sloping part of the dam, the allowable bearing capacity was estimated using (Eq. 2) as shown in Table 7. The analysis results for the allowable bearing capacity of sandstone and limestone units showed a significant variation in the two abutments of the dam. Moreover, there is also a significant variation in allowable bearing capacity for successively stratified rocks at different sections of the dam site which can lead to a potential differential settlement problem.

Conclusion

In this research work, engineering geological investigation of the Chalchal dam site was carried out through engineering geological mapping, discontinuity surveying, core drilling, and in situ permeability testing, geophysical surveying, and laboratory testing in order to evaluate the rock mass quality and geo-mechanical properties and seepage condition. Based on the findings of this study, the following concluding remarks can be made.

- The dam site is underlain by colluvial and alluvial soil deposits, sandstone, limestone with decomposed shale, shale, marly limestone and massive limestone rocks with

Table 7 Estimated allowable bearing capacity of foundation rocks as per Wyllie (1992) and Bell (2007) as cited in Wyllie (1992)

Allowable bearing capacity per Wyllie (1992) using Hoek–Brown failure criteria												
Rock type	Location	UCS (MPa)	GSI	s	m	FOS	q_{ult} (MPa)	q_a (MPa)				
Marly limestone	River Center	27.03	43	0.0018	1.045	3	6.95	2.316				
Shale	Left Abutment	3.96	33	0.0006	0.548	3	0.57	0.19				
Allowable bearing capacity as per Bell as cited in Wyllie (1992) for rocks in sloping parts of the foundation												
Rock Type	Location	C (MPa)	ϕ (°)	Bearing capacity factors			β (°)	H (m)	γ (MN/m ³)	B (m)	FOS	q_a (MPa)
				N_0	N_{cq}	$N_{\gamma q}$						
Limestone	Right Abutment	1.901	27.48	0.309	2.1	0	80	25	0.02352	10	3	1.33
Limestone	Left Abutment	3.006	29.5	0.172	4.9	10.5	20	22	0.02352	10	3	5.321
Sandstone	Right Abutment	0.285	22.4	1.722	3.4	0	45	24	0.02045	10	3	0.1158
Sandstone	Left Abutment	0.375	32.06	1.417	3.2	14.0	40	26	0.02045	10	3	0.877

UCS intact rock strength, GSI Geological Strength Index, s and m rock mass constants, FOS factor of safety, q_{ult} ultimate bearing capacity, q_a allowable bearing capacity, C cohesion, ϕ friction angle, N_0 , N_{cq} and $N_{\gamma q}$ bearing capacity factors, β inclination of slope, γ unit weight of the rock, H height of the slope, B width of foundation, C_{f1} and C_{f2} correction factor. (C_{f1} and C_{f2} = 1 was used for the calculation)

different degrees of weathering and fracturing. These rock units are cut by three dominant joint sets namely NW–SE (JS1), NE–SW (JS2), and ENE–WSW (J3) with the former one being parallel to the river flow direction thereby favoring a potential seepage.

- Detailed field survey and VES conducted along the dam axis have shown the presence of a major fault passing through the right abutment of the dam which poses a serious seepage problem.
- The Lugeon test conducted along the dam axis also revealed potential seepage zones through the left abutment of the dam and river center as the former is underlain by highly permeable rocks with Lugeon value as high as 39.58 and the latter with permeable rock with a Lugeon value of 9.
- The quality of rock mass at the dam site was mainly determined in terms of RQD and RMR. Hence, the rocks of the dam site are characterized by a high degree of fracturing which classified these rocks into very poor to fair RQD classes except for massive limestone that falls into fair to good RQD classes. Based on RMR values, the rocks of the dam site are classified into poor to fair rock classes.
- The geo-mechanical properties of rocks of the dam site such as friction angle, cohesion, and modulus of deformation as determined using Hoek–Brown failure criteria are in ranges of 21.52 to 32.06°, 0.123 to 3.006 MPa, and 0.747 to 9.308 GPa, respectively. The bearing capacity of the dam foundation as determined using empirical method also ranges from 0.116 to 5.321 MPa. From such significant variations in geo-mechanical properties and bearing capacity, it is highly probable that differential settlement might be another major geotechnical problem of this dam site.

Recommendations

From the findings of geological, geotechnical, and geophysical investigations, the following recommendations can be forwarded.

- Colluvial deposit and 18-m-thick alluvial deposit should be removed from the dam axis before the construction of the dam.
- Curtain grouting to the depth of 20 m for the left abutment and 24 m for the river center is also recommended while a detailed water pressure test is suggested to decide the grouting depth for the right abutment of the dam.
- This study also recommends the application of slurry trench or concrete cutoff to replace curtain grout in the parts of the dam axis where shale unit introduces groutability problem due to its decomposition into a fine soil mass.

- The dam site is characterized by a complex geological and structural setup. Hence, it is also recommended that the layout and design of grouting should be reviewed as comprehensive information is available during the detailed design and construction stages.

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Declarations

Competing interests The authors declare no competing interests.

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