

Research Paper

Geological and Geotechnical Assessment of the Bargun Dam Site of Somali Region, Ethiopia

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Abstract

The lack of detailed geological and geotechnical investigations at the proposed Bargun Dam site, which is located in Ethiopia's Shebelle River Basin, poses significant risks to design integrity, safety, and long-term performance. This study presents a comprehensive geological and geotechnical evaluation aimed at ensuring a safe and sustainable dam design. Field investigations, including geological mapping, test pitting, and soil and rock sampling, were followed by laboratory analyses to determine the physical and mechanical properties of foundation materials. Rock mass quality was assessed using the Rock Mass Rating (RMR), Rock Quality Index (Q), Rock Mass Index (RMi), and Hoek–Brown criteria. Slope stability was analyzed kinematically, while bearing capacity (q_u) and permeability (K_s) were estimated using empirical correlations applicable to data-scarce environments. The results indicated RMR, Q, RMi, q_u , and K_s values of 46–59, 1.06, 0.48, 0.55–23.22 MPa, and 2.14×10^{-2} – 3.7×10^{-5} cm/s, respectively. The average rock mass deformation modulus ranged from 5.03 to 9.64 GPa, suggesting a potential risk of differential settlement. Based on the kinematic analysis the left abutment slope is susceptible to oblique toppling, requiring slope modification. However, the site's low seismicity and manageable sediment conditions are favorable for dam construction. Overall, detailed subsurface exploration to identify concealed cavities and targeted grouting are recommended to mitigate leakage and stability risks, ensuring the dam's long-term safety and functionality.

1. Introduction

Nearly 80% of Ethiopia's population lives in rural areas and depends on agriculture for livelihood; thus, rural development a key national priority (OECD/PSI, 2020). However, severe water scarcity, poverty, and food insecurity persist due to rapid population growth, lifestyle changes, and climate variability (World Bank, 2020). Total absolute poverty in rural Ethiopia increased from 23.5 to 33% between 2015/16 and 2021/22, a rise of 9.5% (Degye et al., 2024).

To mitigate drought and enhance food security, the Ethiopian government has constructed several dams across the country. To get sustained economic benefits

from the dams, adequate geological and engineering investigations should be carried out during the planning and pre-construction stages (Yahya et al., 2024). Such investigations are indispensable for ensuring the structural integrity, functionality, and long-term stability of dam projects. Consistent with this, previous studies (Ahmad et al., 2022; Benhammadi et al., 2022; Santa et al., 2021; Adamo et al., 2020; El-Naqa & Al Kuisi, 2002) have emphasized the importance of comprehensive geological and engineering investigations to ensure site safety, dam integrity, and overall stability of water conservation structures.

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Failure to undertake such investigations can result in significant economic losses, and in severe cases, loss of life.

Standard geological and geotechnical investigations for dam foundation assessment involve techniques such as discontinuity mapping, core drilling, geophysical surveys, and laboratory testing (Birhanu, 2025; USBR, 1998). Rock mass characterization commonly relies on discontinuity surveys, point load and uniaxial compression tests, while slope stability is assessed through kinematic, limit equilibrium, and numerical analyses (Laws et al., 2003). Based on these assessments, appropriate mitigation measures, such as grouting, installation of impermeable coatings, rock reinforcement, slope modification, and drainage systems, are implemented to improve dam safety and performance (Hoek et al., 2000; Hoek & Palmieri, 1998).

Hoek and Palmieri (1998) emphasized the necessity of detailed engineering-geological data for assessing slope and foundation stability, including information on rock mass characteristics, faults, discontinuities, structural systems, and groundwater conditions. Similarly, Ahmad et al. (2018) applied integrated geological and geotechnical methods to characterize the Dasu hydropower project site, while Adamo et al. (2020) highlighted the importance of understanding actual foundation conditions before defining treatment methods and design parameters. They further stressed the critical role of close collaboration between geologists and design engineers throughout the dam construction process to mitigate geological hazards. Comparable geo-mechanical characterizations have been conducted for the Marao Tunnel (Portugal), the Tannur Dam site (South Jordan), and the Büyükyenice reservoir area (Turkey), all of which underscore the importance of reliable geological and geotechnical data in dam engineering (El-Naqa & Al Kuisi, 2002; Santa et al., 2021; Tunusluoglu, 2020).

The proposed Bargun Dam project, located within the Shebelle River Basin of Ethiopia, is expected to play a vital role in addressing the region's chronic water scarcity and drought challenges. However, the dam site lies within gypsum and limestone formations, which are prone to dissolution and karst development. These geological characteristics present significant challenges

for dam foundation stability, seepage control, and long-term performance. The insufficient site characterization can lead to design errors, cost overruns, maintenance challenges, and operational risks. Consequently, a comprehensive and detailed geological and geotechnical evaluation is essential to ensure the dam's structural safety and economic feasibility.

Therefore, this study aims to assess the geological and geotechnical conditions of the Bargun Dam site to support the design and construction of a safe, resilient, and cost-effective structure. The study integrates fieldwork, laboratory testing, and empirical analysis to generate reliable baseline data for the project area. It investigates the site's surface and subsurface conditions, rock mass quality, slope stability, seismic setting, and construction material suitability to identify potential geotechnical risks and guide design decisions. The findings not only contribute to the safe construction of the Bargun Dam but also provide a practical methodological framework for similar dam projects in data-scarce and geologically complex regions across the country.

2. Materials and Methods

2.1 Description of the study area

The study area is located around Bargun Village in the Etele District of the Somali National Regional State, southeastern Ethiopia (Figure 1). It lies approximately 355 km southeast of Jigjiga, the regional capital, and about 52 km northeast of Gode Town. A lack of detailed site investigation data and persistent water scarcity remain critical challenges in Ethiopia's Somali Region, particularly in areas like Bargun Village, where recurrent droughts and sudden flash floods severely affect water availability and stability.

2.2 Materials

This study utilized geological hammer, compass, hand lens, GPS device, field books, and topographic map during the field survey and assessment of rock mass discontinuity and rock material strength. The materials used during test pitting, sampling, and field data collections were spade, shovel, trowel, bucket, measuring tape, sack, sample bag, and camera. DIPS 6.0, RocLab and ArcGIS software were used to examine the discontinuity data, to analyze rock strength, to analyze spatial data and create maps, respectively.

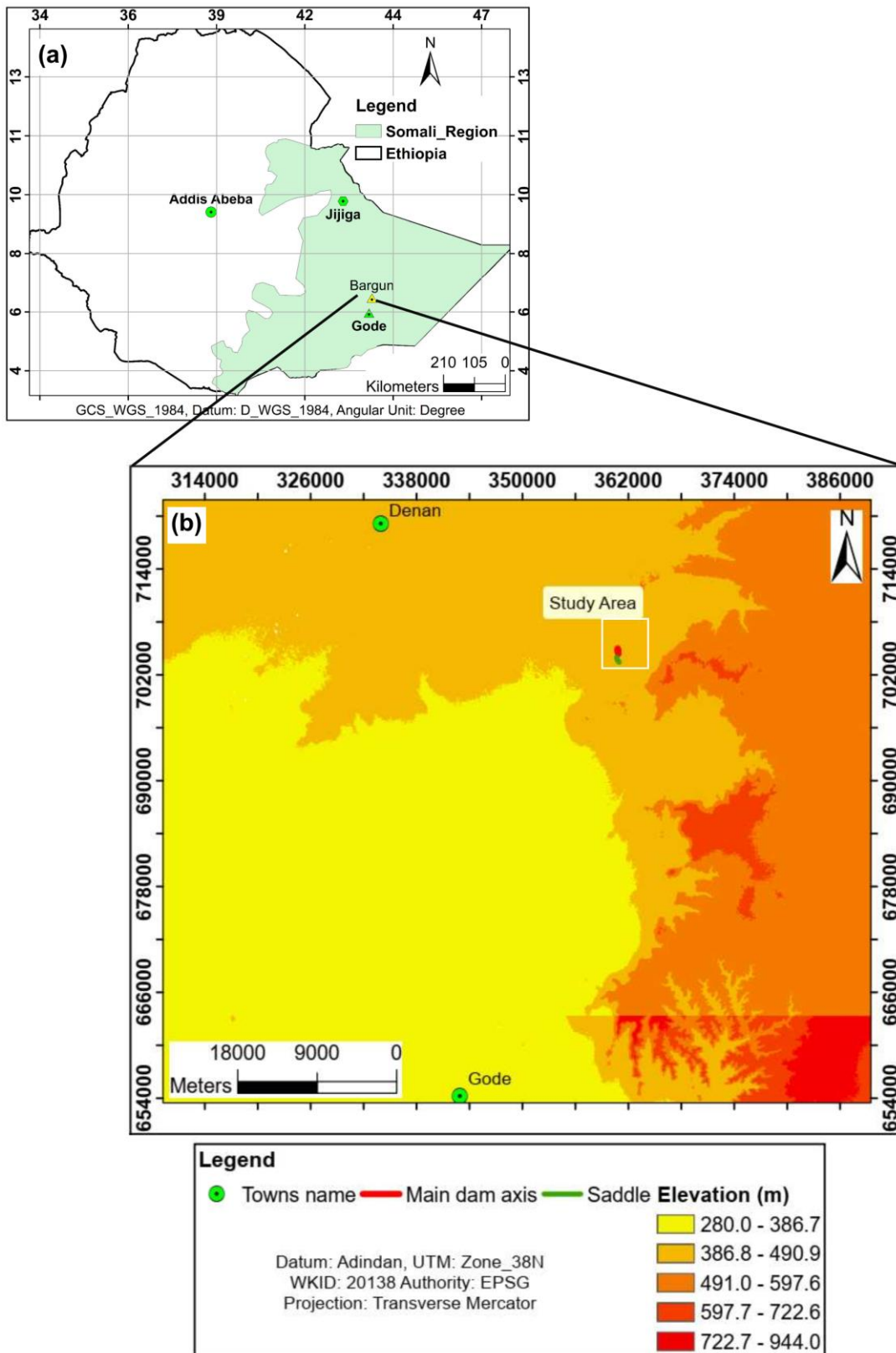


Figure 1: Location map of the study area, (a) location of the study area in the Somali region of Ethiopia, (b) elevation differences around the study area

2.3 Methods

2.3.1 Field and laboratory investigation methods

The field investigation included geological mapping, test pitting, and the collection of both soil and rock samples. Preparation of the geological map for the dam site, saddle, and reservoir area involved detailed documentation of field survey data and interpretation of aerial photographs and satellite images. During the fieldwork, a total of eight test pits (shallow excavations) were conducted to examine subsurface soil conditions and collect samples for laboratory analysis.

Nine rock blocks were also collected, four from carbonate mudstone and five from gypsum rock outcrops within the project area. Laboratory testing of the soil and rock samples was carried out at the Somali Study, Design and Supervision Works Enterprise Laboratory Services. These tests aimed to describe and classify the samples, evaluate the fundamental behavior of the soils and rocks, and determine key geotechnical parameters relevant to the study objectives.

Nine soil samples were analyzed to determine properties such as unit weight (Y), specific gravity (G_s), particle size distribution (gravel, sand, and fines), liquid limit (LL), plastic limit (PL), plasticity index (PI), free swell, optimum moisture content (OMC), maximum dry density (MDD), saturated hydraulic conductivity (K_s), cohesion (c), and internal friction angle (ϕ).

The analysis of two rock samples included determination of their unit weight (Y), specific gravity (G_s), water absorption, Aggregate Impact Value (AIV), and soundness using sodium sulfate (SSS). All laboratory tests were conducted in accordance with the relevant ASTM and BS standards. The activity (A) values of the fine-grained soils were calculated using the formula proposed by Skempton (1953):

$$Activity = \frac{Plasticity\ Index}{\% \text{ finer than } 2\mu m} \quad (1)$$

Clays are classified into inactive, normal and active clays, corresponding to activity values of less than 0.75, 0.75 to 1.25 and greater than 1.25, respectively (Birhanu & Vishal, 2020).

2.3.2 Rock mass indices, deformation modulus and kinematic analysis

Rock Mass Indices: The rock mass properties of the

dam foundation and abutment slopes were analyzed based on the rock mass classification systems of the Rock Quality Designation, rock mass rating, Rock Tunnelling Quality Index, rock mass index, Geological strength index, and the Hoek-Brown failure criterion.

Rock Quality Designation (RQD) is the ratio of the sum of the lengths of all core pieces greater than 10 cm to the total length of the core run. Unfortunately, there were no historical logs of RQD from the present study area or a core drilling program. Therefore, this study followed an alternate method of estimating RQD, described by Hutchinson & Diederichs (1996), by placing a two-meter graded rule on an exposed rock face and calculating RQD as described for the drill core, considering any well-developed joint which intersects the ruler as a core break.

The RQD values were used as input in the rock mass rating (RMR) calculation, based on Bieniawski (1989), given by:

$$RMR = A_1 + A_2 + A_3 + A_4 + A_5 + B \quad (2)$$

where A_1 , A_2 , A_3 , A_4 , A_5 and B represent ratings for the uniaxial compressive strength of the rock material, the RQD, the spacing of joints, the condition of joints, ground water conditions, and the orientation of joints, respectively.

The Tunnelling Quality Index (Q system) was determined by eq. (3) (Barton et al., 1974).

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad (3)$$

where RQD is given as the value for this parameter and J_n , J_r , J_a , J_w and SRF are ratings for the number of joint sets, the joint roughness, the joint alteration, the joint or ground water, and the rock mass stress situation, respectively.

The rock mass index (RMi) is a volumetric parameter indicating approximate uniaxial compressive strength of a rock mass. Equation (4) was used to determine the RMi:

$$RMi = \sigma_c * JP \quad (4)$$

where σ_c is uniaxial compressive strength of the intact rock, JP is the jointing parameter.

The geological strength index (GSI) relied on the

rock mass's visual appearance and structural characteristics. However, this study utilized the 1989 version of RMR to determine GSI, as proposed by Hoek et al. (2000), and given as Eq. (5):

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

Deformation modulus (Em): for all values of RMR, and R_{Mi}, Serafim & Pereira (1983) formula (eq. (6)) and Palmstrom (2009) formula (eq. (7)), respectively, were used in the estimation of the corresponding Em values.

$$Em \text{ (GPa)} = 10^{(RMR-10)/40} \tag{6}$$

$$Em = 7 \times RMi^{(0.4)} \tag{7}$$

Kinematic Analysis of the Rock Slopes: This study comprehensively assessed the discontinuities within the dam's left abutment slope (LAS) and right-abutment slope (RAS) to gather pertinent data regarding their parameters. Using the geological structures' orientation data, the study assessed kinematic analysis of the slope stability condition of the RAS and LAS of the dam site for the planar, wedge, and toppling modes of failures.

2.3.3 Bearing capacity, permeability and seismicity estimation

Foundation bearing capacity: While Terzaghi's, Meyerhof's, Hansen's, and Vesic's methods are the most popular shallow foundations' bearing capacity calculation methods, Vesic's N-factors are generally prevalent in the design codes worldwide. Considering the effect of embedment depth, the bearing capacity formula included in the various methods and design codes takes the following form:

$$q_u = cN_c d_c + \frac{1}{2} \gamma B N_\gamma d_\gamma + \gamma D_f N_q d_q \tag{8}$$

where: q_u is the ultimate bearing capacity, often referred to just as the bearing capacity, c and γ are cohesion and unit weight of the soil, B is the width of the strip footing, N_c , N_γ , and N_q , are the three N-factors for the cohesion, the weight of the triangular soil wedge, and the lateral surcharge term, d_c , d_γ and d_q are the depth factors for the cohesion, the weight of the

triangular soil wedge, and the lateral surcharge term respectively.

The bearing capacity of the Bargun dam foundation was estimated using equation (8), with inputs given below:

1. For flood plain (silty clay) deposits, the average values of shear strength are $c=23$ kPa, $\phi = 17.4$, and unit weight = 13.95 kPa and from the Vesic (1973) $N_c=13.175$, $N_q=5.425$, $N_\gamma=4.425$, for $B=D_f=1$, $d_c=1.4$, $d_q = 1 + 2xtan(\phi)x(1 - \sin(\phi))^2$, and $d_\gamma=1$.
2. For residual soil deposits, the average values of shear strength are $c=10$ kPa, $\phi = 25$, and unit weight = 14.13 kPa and from the Vesic (1973) $N_c=22.45$, $N_q=12.4$, $N_\gamma=13.95$, for $B=D_f=1$, $d_c=1.4$, $d_q = 1 + 2xtan(\phi)x(1 - \sin(\phi))^2$, and $d_\gamma=1$.
3. Since the channel deposits are thin layers with prominent permeability and are undesirable as foundation material, their bearing capacity was not estimated.

Rock mass bearing capacity was estimated using the relationships proposed by Bell (1992), Kulhawy & Carter (1992) and Wyllie (2003), given below as equations (9), to (11), respectively.

$$q_u = \sigma_{ci}(s^{0.5} + (m_b s^{0.5} + s)^{0.5}) \tag{9}$$

$$q_u = \sigma_{ci}(s^a + (m_b s^a + s)^a) \tag{10}$$

$$q_u = \sigma_{ci} C_{fi} s^{0.5} (1 + (m_b s^{-0.5} + 1)^{0.5}) \tag{11}$$

The uniaxial compressive strength of intact rock σ_{ci} in MPa, m_b , s and a are the Hoek-Brown constants, and C_{fi} is the correction factor for foundation shapes (taken as 1.25 for a square pier).

Foundation permeability: Permeability of the soil material at the dam foundation was assessed based on the laboratory examinations of the saturated hydraulic conductivity values of the collected soil samples. However, the rock masses' permeability condition depends on the fractures in fractured rock masses. The fluid flow through the intact rock samples of these rocks is usually so low that significant fluid movement can only occur through the fractures (El-Naqa, 2001). Thus, a method involving fracture characteristics was used to

characterize the hydraulic conductivity of such rock masses at the dam site. El-Naqa (2001) proposed Eq. (12) based on relationships between the hydraulic conductivity, K and the RMR from field mapping of fractures as presented below:

$$K = 3166.1xeEXP^{(-0.0755xRMR)} \tag{12}$$

Seismicity hazard: Seismicity analysis of the dam site was undertaken based on Ethiopia's global seismic hazard profile map.

3. Results and Discussion

3.1 Geology of the dam site

The primary geological formation in the study area is Korahe (main gypsum) formation as shown in Figure 2. The regional geological structures have NE-SW and NW-SE trends. While no significant regional structures are present locally, minor joints characterized by specific orientations and spacing are indeed observed and documented. These joints are considered local structural features observed in outcrops of gypsum rocks at the dam's right and left abutment slopes (Table 1). The local geological units are, (1) quaternary (recent local deposits) that include alluvial, colluvial and slope deposits. Texturally, they are sands, silts and clays. (2) Main gypsum formation with rock types gypsum and carbonate mudstone (mud/ limestone). The central dam axis crosses the mud/ limestone and gypsum rocks and residual soils at the RAS, alluvium (flood plain and channel deposits) in the central river valley, and gypsum rock at the LAS. The saddle dam site lies within gypsum rock masses, whereas the reservoir area displays recent deposits of sands with gravel (channel deposit), clays with silt (flood plain deposit), residual soils and gypsum rock masses.

The central river valley (bed) and abutments in general comprise of five geological units, namely, channel deposit (alluvial), floodplain deposit (alluvial),

residual soil, lime/mudstone and gypsum rock masses. Figure 2 shows a surficial view of the dam sites' geological (geotechnical) conditions and reservoir area.

The alluvial deposits are remarkably heterogeneous and consist of alternating silt, clay and sand of various colours and origins (Figure 3a-c). The sandy or scarcely clayey yellow and grey deposits are weathered materials from the high crystalline or volcanic plateaus. In contrast, the red, brown, and typically more clayey deposits originate from the weathered limestone or gypsum rocks (Figure 3a). The local tributaries brought the deposits during the filling in of the valley. The residual soils (weathered layers) are red decalcification clay on lime/mudstone and gypsum crusts on gypsum formations (Figure 3c).

3.2 Characterization and orientation of discontinuities

The findings, given in Table 1 and illustrated in Figure 4, explain the key attributes of major discontinuity sets and present a rosette diagram depicting the measurements obtained from both abutments. The discontinuities revealed that the right abutment exhibited two primary joints characterized by strike directions of NNW–SSE (J1R), and NEE–SWW (J2R). In contrast, the left abutment exhibited striking joints in the NWW–SEE (J1L), NW–SE (J2L), and NNW-SSE (J3L) directions. These observations suggest that J2R of the right abutment and J1L of the left abutment possess orientations conducive to leakage through the discontinuities because their strike directions' slight alignment with the river's flow direction at the respective abutments. This phenomenon raises concerns about potential water seepage. Consequently, it is imperative to incorporate specific measures in both abutments during the construction phase to mitigate the risk of potential leakage.

Table 1: Summary of representative discontinuity characteristics

Location	Joint set	Spacing (cm)	Persistence (m)	Aperture (mm)	Roughness	Infilling	Degree of weathering
Left abutment	J1	25	1.8	15	Slightly Rough	Sand/Silt	High
	J2	40	1.3	10	Slightly Rough	Sand/Silt	Moderate
	J3	50	1.5	8	Slightly Rough	Sand/Silt	Moderate
Right abutment	J1	20	0.4	15	Rough	Hard Clay	Moderate
	J2	10	0.5	20	Rough	Hard Clay	High

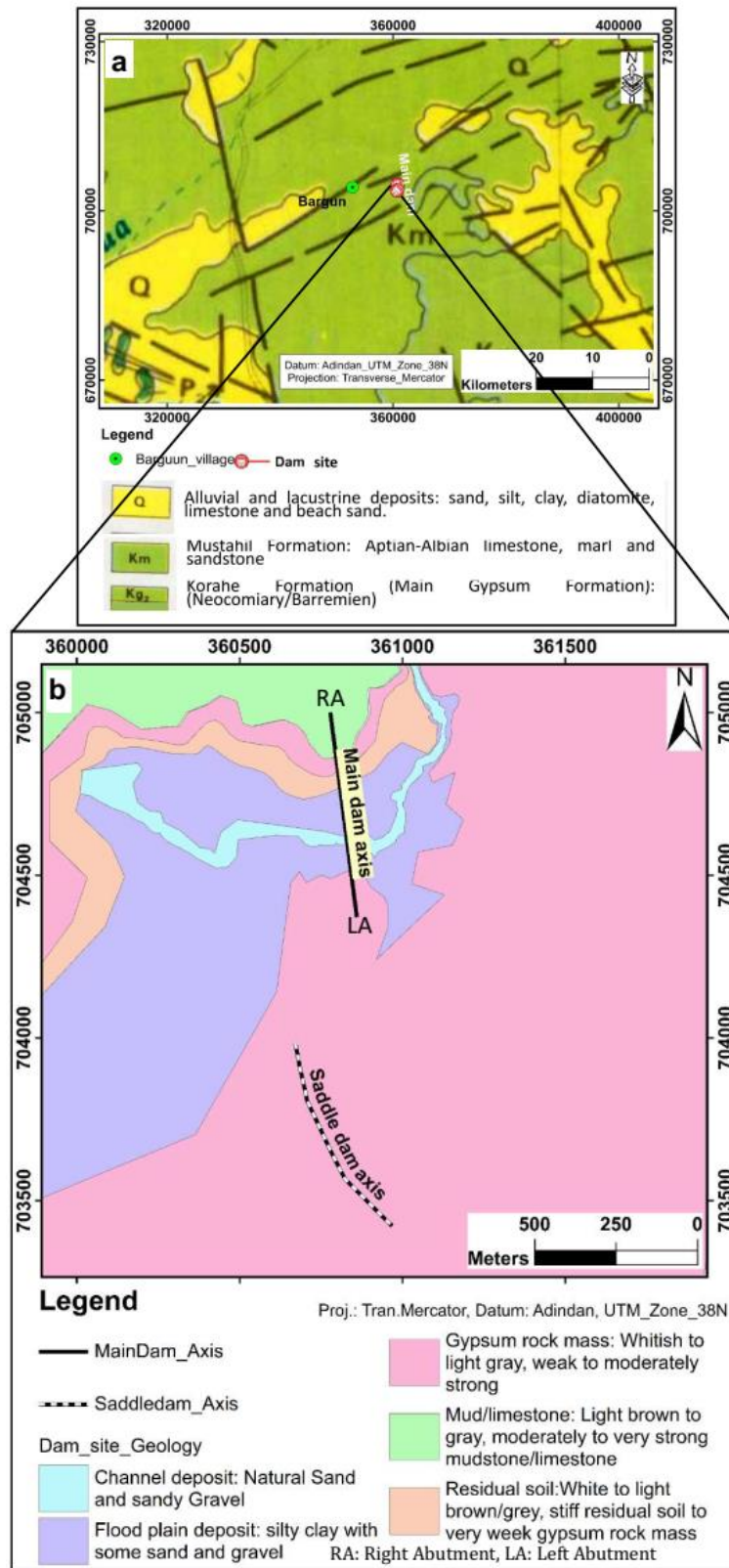


Figure 2: (a) Regional geological map of the proposed main and saddle dam sites compiled from geological map of Ethiopia @ scale 1:2,000,000 and (b) Engineering geological map of the study area



Figure 3: Photo showing the five geotechnical units identified: **(a)** Flood plain deposits (silty clay) at the RAS, **a'** Flood plain deposits at the LAS side of central valley; **(b)** channel deposit at the main dam central valley (sandy Gravel), **b'** Sand; **(c)** residual soil (completely weathered rock) at the RAS, **c'** residual soil at the center of the saddle axis; **(d)** gypsum rock mass at the LAS; **(e)** limestone/mudstone rock mass at the RAS upper margin

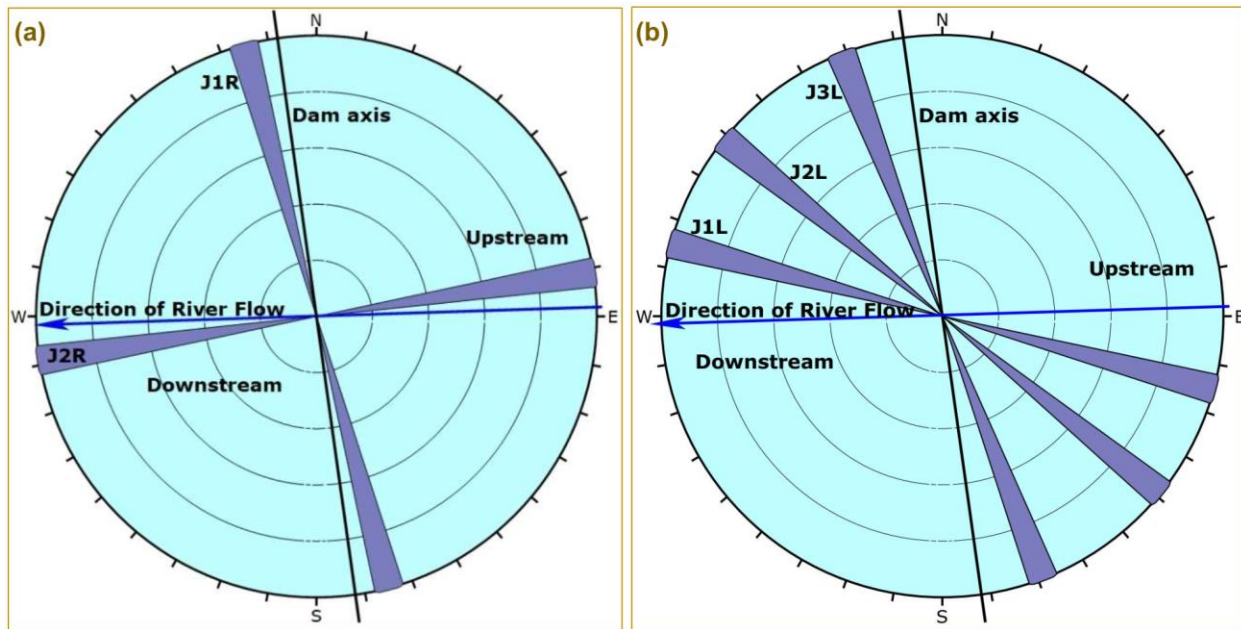


Figure 4: Rosette diagram of discontinuities with respect to the dam axis and the river flow direction, (a) right abutment slopes (b) left abutment slopes (J1, J2, and J3 are joint set 1, 2, and 3, respectively)

3.3 Lab examination of soils and rock samples

The analyzed lab test results of the soil and rock samples collected during field work are given in Tables 2 and 3, respectively. The results in Table 2 show that the soil materials (BDRivS1, BDLTP1, BDRTP2, and BDRTP1) have properties such as average permeability (K_s) equal to 1×10^{-5} cm/s, and $PI = 13.76-35\%$, indicating unsuitable foundation material due to its high plasticity. The permeability of the natural sand and sandy gravel materials (BDRivCL2 and BDCITP4) is 1×10^{-3} cm/s, showing that it is a free-draining nature, suitable for filter materials.

The clay and sand materials are unsuitable as foundation materials due to the high plasticity of the clays and high permeability of the sands overlying gypsum rock with lower permeability. Figure 5 shows that the uniaxial compressive strength value is greater than 38 MPa. Generally, any rock with a compressive strength of 30 MPa or more is satisfactory for rock fill materials. Moreover, the aggregate impact values are within the range of 20 to 25 %, indicating that the results are within permissible impact value limits for water-bound macadam suitable for concrete or asphalt concrete. The low porosity and water absorption values indicate that the rock materials can be a suitable source for concrete.

3.4 Engineering geological units

Figure 6 shows that the foundation and abutments of the dam site are characterized by five engineering geological units (EGU) (from RAS of the main dam towards LAS along the dam axis). EGU-1 is moderately to very strong limestone rock mass exposed at the top layer on the outer margins of the right-abutment slope. This unit displays a light brown to grey, moderately-very strong, limestone rock interlayered with mudstone rock with uniaxial compressive strength of the intact rock (based on experimental field test) varying from 50-100 MPa, and unit weight of 1.46 (g/cc).

EGU-2 is weak to moderately strong gypsum rock mass, which covers most of the dam foundation areas and reservoir area and underlies the limestone unit, residual soil unit and the alluvial deposits. This unit's geotechnical category is the GTU-2. Whitish to light gray, weak to moderately strong gypsum rock mass with UCS of intact rock 38-63 (MPa), unit weight 1.52 (g/cc), RQD 20.5-75, RMR 46-59, cohesion 2.05-2.71 (MPa), friction angle 27.86-31.98 (deg.), ultimate bearing capacity (q_u) 16.07-23.56 (MPa), and hydraulic conductivity 36.81-98.22 Lugeons (Lu).

EGU-3 determined at the dam foundation is the residual soil (weathered gypsum layer). This unit is whitish grey to light brown, stiff residual soil to very weak (weathered) gypsum layer with unit weight of 1.41

g/cc, cohesion 10 MPa, friction angle 25 (deg.), ultimate bearing capacity (q_u) of 0.64MPa and hydraulic conductivity K value of 143 lugeons (Lu). This unit is exposed mainly at the right abutment side of the river valley; however, a skinny layer of residual soil covers along the saddle dam axis.

The floodplain deposit and channel deposit are the fourth and fifth EGU discovered at the dam site. EGU-4 unit is described as light brown to brown, dominantly fine-grained, medium to highly plastic, silty Clay with

some interlayers of sand and gravel. The unit has a unit weight of 1.395 g/cc, cohesion of 23 kPa, friction angle of 17.4 (deg.), ultimate bearing capacity (q_u) of 0.55 MPa, and hydraulic conductivity Ks values ranging between 3-19 lugeons (Lu). EGU-5, exposed at the dam central valley, is characterized by the light brown to grey colour, medium to coarse grained, natural Sand and sandy Gravel with unit weight values of 1.555 (g/cc), and hydraulic conductivity K values of 104 lugeons (Lu).

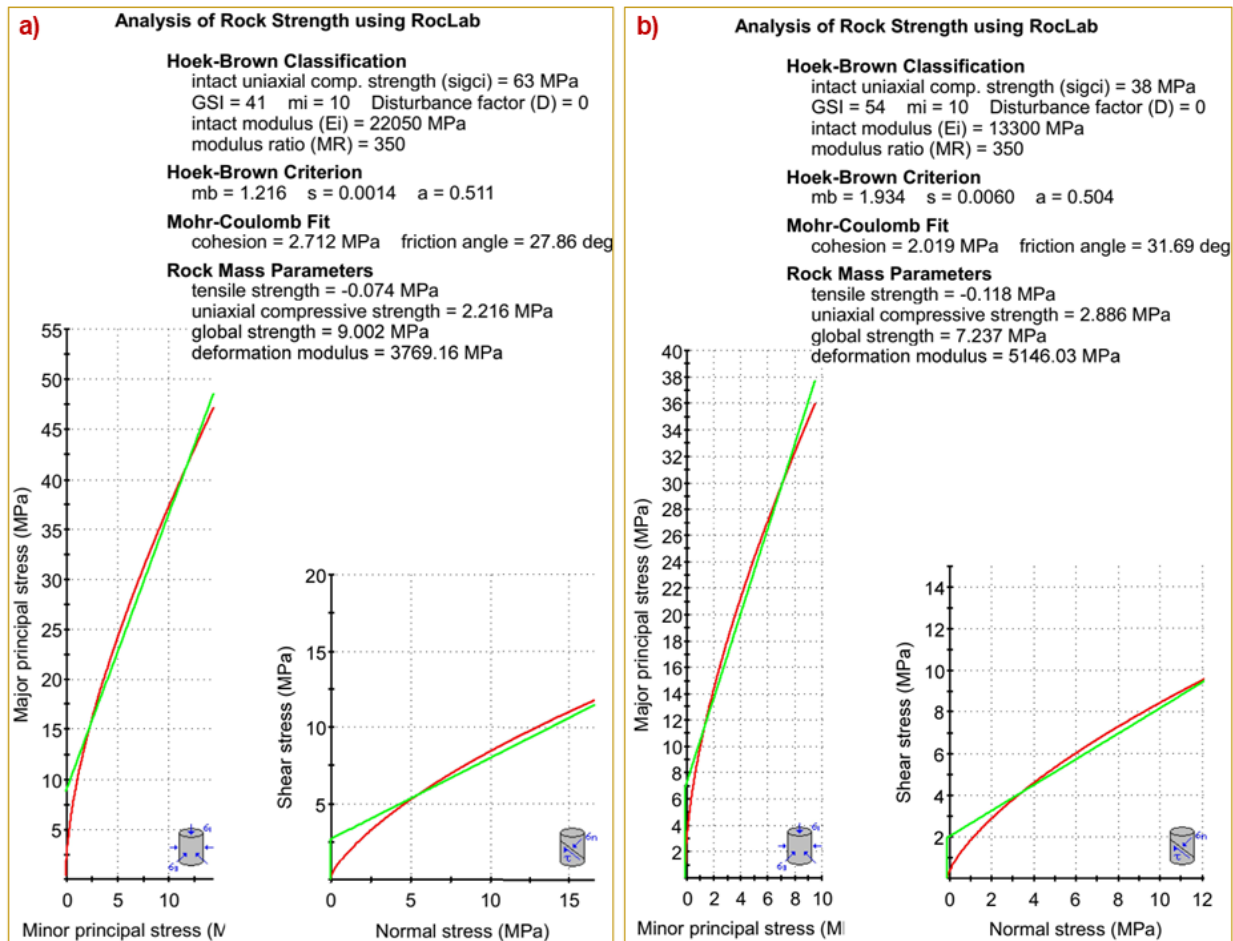


Figure 5: Analysis of gypsum rock strength at a) RAS, b) LAS

Table 2: Laboratory test results for soil samples collected from Bargun Dam site

No.	Sample I.D.	Sampling Depth (m)	UTM Coordinate		Laboratory test type														
			Easting	Northing	Unit weight Y (gm/cm ³)	Specific Gravity	Gravel (%)	Sand (%)	Silt & Clay (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Activity (silt & clay/PI)	Free Swell (%)	Optimum Moisture Content OMC (%)	Maximum Dry Density MDD (g/cm ³)	Permeability Ks (cm/s)	Cohesion c (Kpa)	Internal friction Ø(deg)
1	BDRivS1	0.9 - 1.7	360745.0	704655.0	1.386	2.67	0.5	36.5	63.0	57	21.2	35.3	0.56	42	17.22	1.74	2.50E-05	30	16
3	BDRivS1	0.4 - 2.0	360745.0	704655.0	1.412	2.68	2.9	37.6	51.3	-	-	-	-	29	18.51	1.81	8.42E-04	23	18
2	BDLTP1	0.4 - 0.8	360842.6	704614.3	1.352	2.66	1.8	30.6	67.6	57	24.5	32.5	0.48	43	19.57	1.62	3.70E-05	38	9
4	BSDTP1	0.2 - 0.3	360808.9	703672.9	1.410	2.70	65	22.7	12.0	1	-	-	-	18	12.42	1.98	2.14E-02	10	25
5	BDRivCL-1	2.0 - 3.0	360745.0	704655.0	-	-	-	-	-	35.8	20.9	14.9	-	-	-	-	-	-	-
6	BDRivCL2	0.1 0.9	360842.6	704614.3	1.430	2.70	69	21.9	9.0	-	-	-	-	-	-	-	1.00E-03	-	-
7	BDCITP4	0.1 0.4	360840.0	704671.0	1.680	2.80	-	-	-	-	-	-	-	-	-	-	-	-	-
8	BDRTP1	0.2 0.4	360803.7	704844.7	1.350	2.67	8.6	69.5	21.9	34	20.5	13.8	0.63	-	16.33	1.63	2.35E-04	5	25
9	BDRTP2	0.4 - 1.0	360827.6	704722.1	1.347	2.66	0.7	30	69.3	57	23.7	33.6	0.49	-	18.72	1.58	2.40E-05	19	19

Table 3: Laboratory test results for rock samples collected from dam and quarry sites

No.	Sample I.D.	Sampling Depth (m)	UTM Coordinate		Laboratory test type				
			Easting	Northing	Unit weight Y (gm/cm ³)	Specific Gravity	Water Absorption (%)	Aggregate Impact Value AIV (%)	Soundness by Sodium Sulphate SSS (%)
1	BQ1S1	Surface	354977	702534	1.46	2.67	0.76	24.8	7.1
2	BDLTP3	0.1-0.3	360860	704504	1.52	2.77	0.62	20.7	6.4

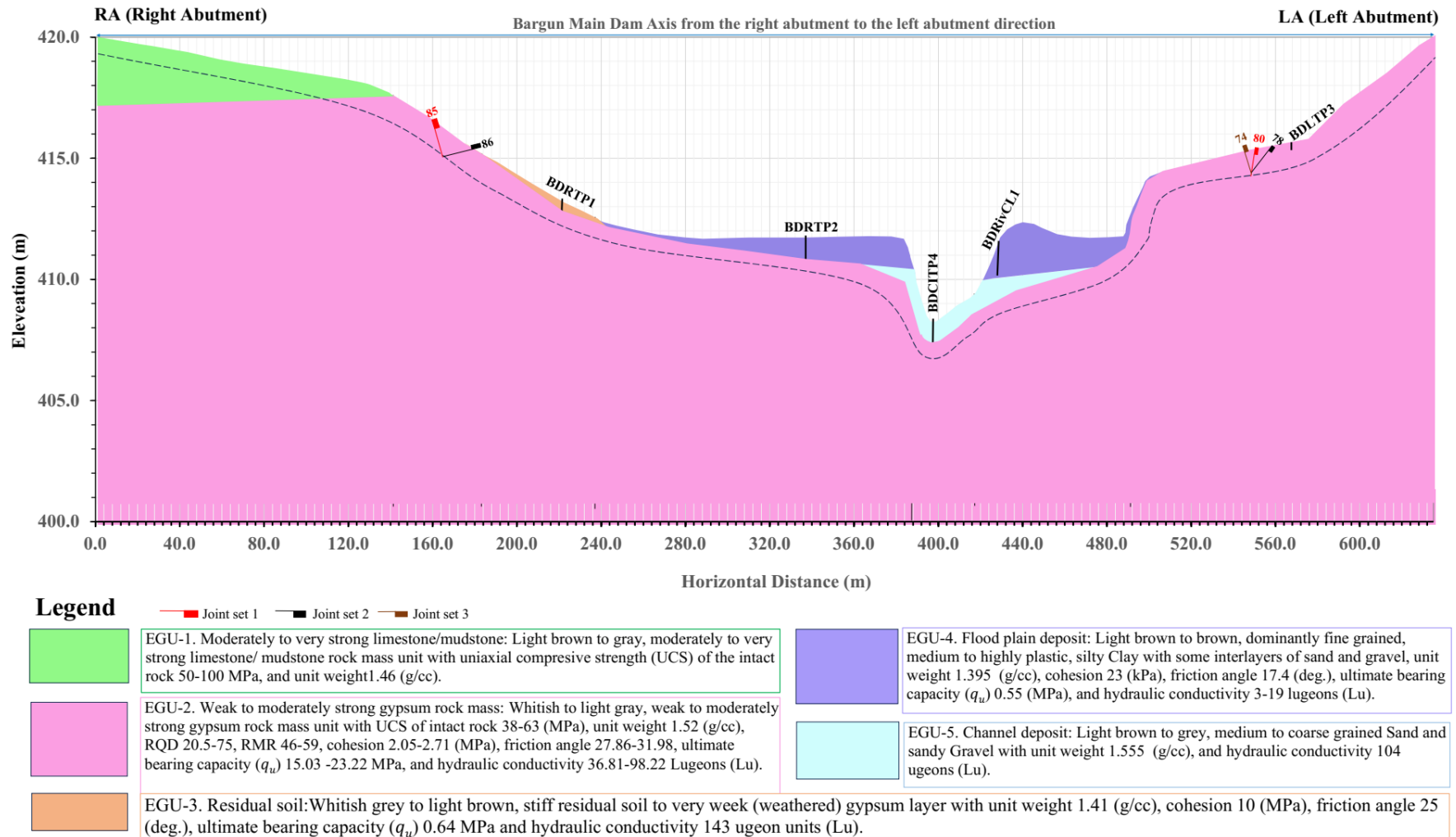


Figure 6: Engineering geological cross section of the main dam foundation from right to left abutment with test pit and joint set locations

Table 4 shows the result of geomechanical investigation. The average RMR vary from 59 at LAS to 46 at RAS, indicating fair rock (rock mass class III). The Q value of 1.06, average of LAS and RAS, shows poor rock mass quality, and the R_{Mi} of 0.48, average of LAS and RAS, entails inferior rock mass. The rock mass index values are acceptable for the dam foundation. However, the average rock mass deformation modulus, E_m, indicate the risk of differential settlement.

Table 4: Geo-mechanical characteristics

Rock mass parameters	LAS	RAS	Av.	
RMR	59.0	46.0	52.5	
Q	1.944	0.175	1.06	
R _{Mi}	0.661	0.301	0.48	
E _m (GPa)	Serafim & Pereira (1983)	18.00	7.94	12.97
	Palmstrom (2009)	5.93	4.33	5.13
	Rocscience (2007)	5.15	3.17	4.16
	Average	9.69	5.15	7.42

3.5 Foundation stress capacity

For the dam's safety, the structural load should not exceed the ultimate bearing capacity (q_u) of the rock masses at the foundation. In this study, bearing potentials of the residual and flood plain deposits at the dam site are given in Table 5. The average q_u of the residual and flood plain deposits is 0.643 and 0.554 MPa for the residual at RAS and flood plain deposits at the central valley area, respectively. The load allowable is 0.21 MPa at the RAS (for residual soil) and 0.18 MPa at the central valley (for flood plain deposits) based on the

average q_a values.

Table 5: Bearing capacity of the residual and alluvial (flood plain) soils at the dam foundation

Parameters	Residual soil at RAS (EGU-3)	Flood plain deposit at central valley (EGU-4)
Input parameters		
c (KPa)	10.000	23.000
Ø (deg)	25.000	17.400
Y (KPa)	14.130	13.950
N _c	22.450	13.175
N _q	12.400	5.425
N _y	14.000	4.425
d _c	1.400	1.400
d _q	1.311	1.308
d _y	1.000	1.000
B=Df	1.000	1.000
Determined parameters		
¹ q _u (MPa)	0.643	0.554
FOS	3.00	3.000
² q _a (MPa)	0.21	0.180

¹q_u: ultimate bearing capacity; ²q_a: allowable bearing capacity

Table 6 presents load potentials of the gypsum rock masses at the dam site. The result indicates that the q_u of gypsum rock varies from 15.03 at the RAS to 23.22 MPa at LAS, while the average q_u are 20.02 and 17.06 MPa at LAS and RAS, respectively. This shows that the load allowable for the gypsum rock masses at the LAS is 6.67 MPa and at the RAS is 5.69 MPa based on the average of q_a values. The study further revealed that the analysis results of the allowable bearing capacities of the soil material differed, with the lowest q_a value of 0.18 MPa (at the central valley), and a prominent q_a value of 6.67 MPa (at the LAS) (Tables 5 and 6 and Figure 5).

Table 6: Load bearing potential of gypsum rock masses at the LAS and the RAS of main dam foundation

Parameters	Load bearing capacity of gypsum rock at LAS			Load bearing capacity of gypsum rock at RAS		
	Bell (1992)	Kulhawy & Carter (1992)	Wyllie (2003)	Bell (1992)	Kulhawy & Carter (1992)	Wyllie (2003)
UCS (MPa)	38	38	38	38	38	63
m _b	2.005	2.0046	1.934	1.934	1.934	1.216
s	0.007	0.007	0.007	0.007	0.007	0.001
a	0.504	0.504	0.504	0.504	0.504	0.511
Cf ₀	-	-	-	-	1.250	-
¹ q _u (MPa)	18.58	18.25	23.22	16.07	15.03	20.09
Mean		20.02			17.06	
FOS	3.00	3.00	3.00	3.00	3.00	3.00
² q _a (MPa)	6.19	6.08	7.74	5.36	5.01	6.70
Mean		6.67			5.69	

¹q_u is the ultimate bearing capacity ²q_a is the allowable bearing capacity

3.6 The foundation permeability, slope stability and seismic hazard

Foundation permeability: Table 2 shows that the alluvium (channel deposit) at the central river valley of the dam foundation has permeability 1.04×10^{-3} cm/s (BDRivCL-1), indicating good drainage (pervious) nature (USBR, 1998). This deposit is unsuitable for dam foundation because its permeability is higher than that of the underlying gypsum rock; however, it can serve as the drainage, filter, and aggregate material for dam construction. On the LAS of the valley slopes, the flood plain deposits' permeability values vary from 8.42×10^{-4} cm/s (BDRIVB1) to 2.5×10^{-5} cm/s (BDRivS1), indicating good drainage to poor drainage (semi-pervious) nature (USBR, 1998). On the RAS, the flood plain deposits' permeability values vary from 3.5×10^{-5} cm/s (BDRTP1) to 2.4×10^{-5} cm/s (BDRTP2) indicating poor drainage nature (USBR, 1998) (Table 2 and Figure 2). Considering the low thickness of the alluvial deposit in the dam foundation area (Figure 2), the dam should base on bedrock since removing sediment from under the dam's base in this case will not present any particular difficulties.

The Main-Gypsum-Formation outcrops over large parts of the study area (Figures 2 and 5), which consists of gypsum and limestone with interbedded mudstones. Based on the results of Table 7, the permeability values more prominent on the RAS than in the LAS. This result is in agreement with the fact that the joint sets in the RAS have lower spacing (10-20 cm), and higher aperture (15-20 mm) than the joint sets in the LAS with spacing of 25-50 cm and aperture of 8-15 mm, which causes RAS more permeable than the LAS (Table 1).

Furthermore, the study result shows that the lugeon (Lu) values are within the five-unit changes (15-50 range) on the LAS and ten-unit changes (50-100 range) on the RAS for equal significance (Houlsby, 1976), indicating values more significant than recommended (7-10 Lu for earth/rock dams with wide cores and three rows of curtain grouting, and 5-7 Lu for concrete dams

with three rows of curtain grouting). On the other hand, the geological maps (Figures 2 and 5) show that the Bargun dam site is mainly free from significant geological risks associated with faults and buried channels. Although soluble rocks like gypsum and limestone are present, the author did not encounter any signs of cavities or sinkholes, suggesting minimal impact on reservoir stability and water tightness.

Moreover, the permeability condition of the reservoir depends on the primary gypsum formation, containing interstratified confined aquifers with a very low transmissivity. Owing to the impervious nature of the anhydride, the gypsum in the reservoir area is likely a geological formation with low permeability and porosity (Quiroga et al., 2022), suggesting the feasibility of the reservoir for the intended dam project.

Slope stability: This study assessed the kinematic analysis of the rock slope stability condition of the RAS and LAS of the Bargun dam site for the planar, wedge, and toppling modes of failures and the results are shown in Figures 7. Figure 7a indicates that the major joint set creates a critical condition for planar failure on the RAS. Figure 7b reveals that none of the three major joint sets creates a critical condition for planar failure on the LAS. In Figure 7c, the one joint intersection analyzed has not created a critical condition for wedge mode failure. Similarly, Figure 7d indicates that none of the three joint intersections created was critical to cause wedge failure on the LAS. The kinematic analysis result indicates that the RAS is free of toppling modes of failures (Figure 7e). However, the LAS is subject to oblique toppling mode failures due to two joint intersections that can cause an oblique toppling mode of failure.

In general, the result of kinematic analysis indicates that both RAS and LAS are free from planar and wedge mode failures. While the RAS is free from toppling mode failures, LAS is susceptible to oblique toppling mode failures due to the intersection of J1 and J3 and J1 and J2. Thus, slope modification needs consideration to address the toppling failures at the LAS.

Table 7: Permeability of the Gypsum rock masses at the LAS and RAS of Bargun Dam site

Parameters	LAS	Remarks	RAS	Remarks
RMR values	59	Fair rock mass class III	46	Fair rock mass class III
Ks (cm/s)	3.7×10^{-4}	Poor drainage (semi-pervious)	9.8×10^{-4}	Poor drainage (semi-pervious)
Ks (LU)	37	Lu (37): 15 to 50, 5 units	98	Lu (98): 50 to 100, 10 units

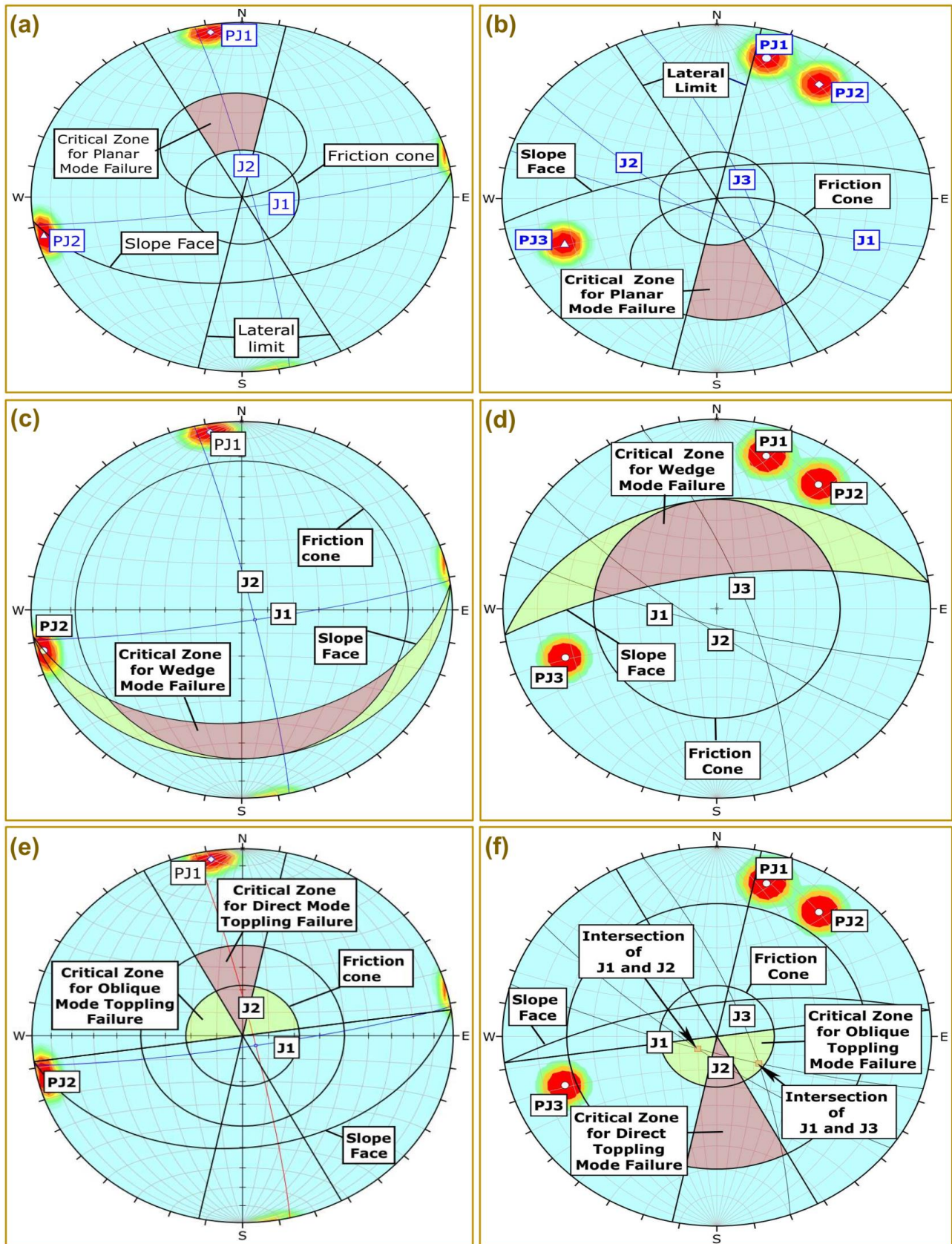


Figure 7: Kinematic analysis of slope stability, (a) RAS Planar Mode Failure check, (b) LAS Planar Mode Failure check, (c) RAS Wedge Mode Failure check, (d) LAS Wedge Mode Failure check, (e) RAS Toppling Mode Failure check, and (f) LAS Toppling Mode Failure check

Seismic hazard: The East African Rift and Afar Depression are seismically active, requiring earthquake-resistant infrastructure. Based on Silva et al. (2018), the study area falls within a seismic zone with a peak ground acceleration of 0.09 g for a 475-year return period. Seismic zones with a design ground acceleration

not greater than 0.05 g are low seismic zones, for which reduced or simplified seismic design procedures for specific types or categories of structures may be used. The result shown in Figure 8 shows low seismic hazard at the Bargun dam site.

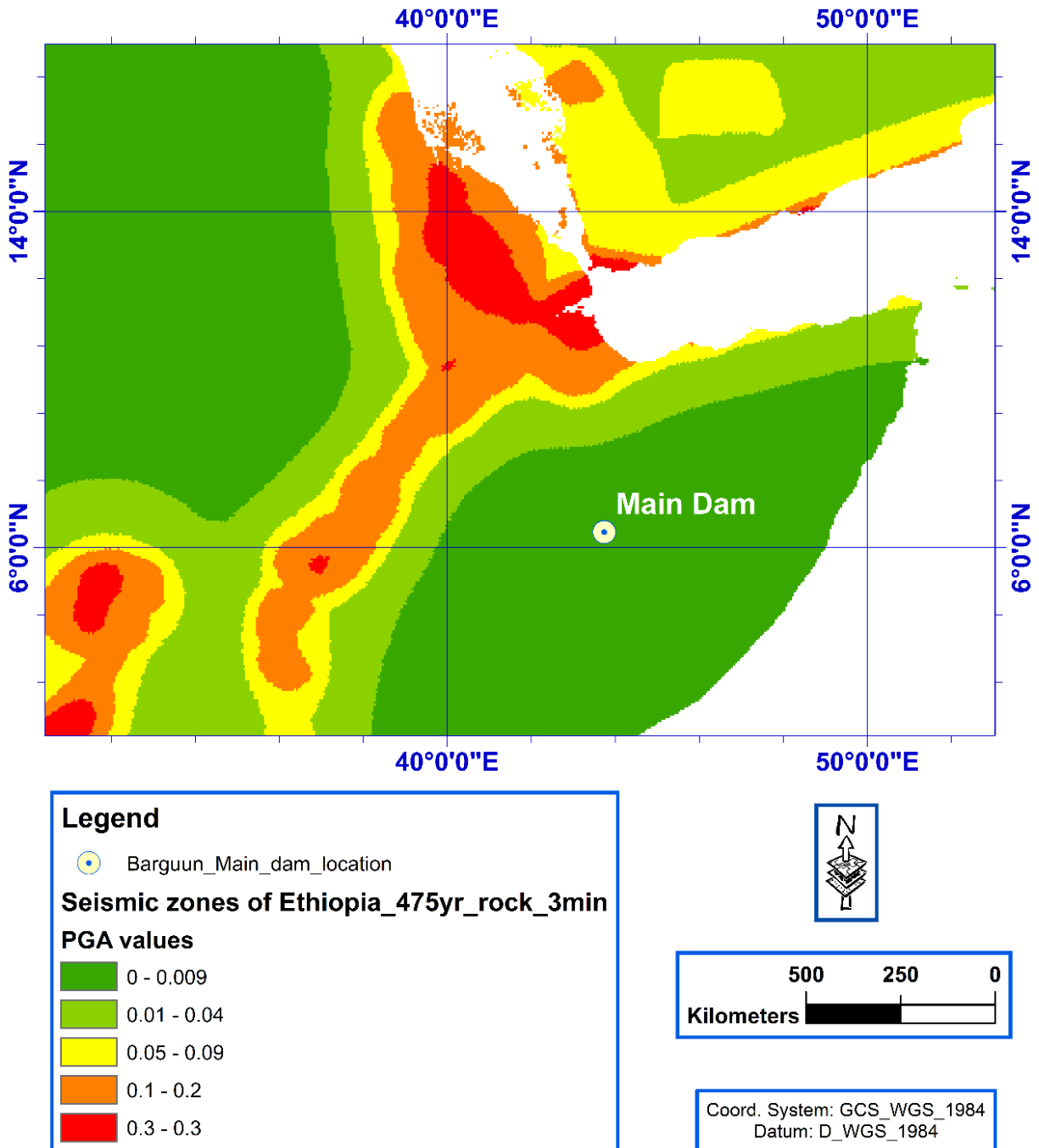


Figure 8: Seismic hazard map of the Bargun dam site

Subsurface cavities and sinkholes: The presence of rocks such as limestones and gypsum at the dam sites and reservoir area indicates the risk of dissolution and cavity creation (Cooper & Gutiérrez, 2013; Sissakian et al., 2017). However, incongruent with findings of previous studies by the Natural Resources Management Commission (NRMC) of Ethiopia, no manifestation of sinkholes, karsts and underground cavities were observed in the project site during the present study. According to NRMC (1972), boreholes drilled at about 32, 54 and 72 km distances from the present study area at Danan (110 m deep), Gode (250 m deep), and Kebri-Dahar (212 m deep), respectively, showed no underground cavities. Furthermore, NRMC (1974) indicated that dissolution indices found in the boreholes at the Kuldash dam project in the Wabisheble basin are either small cavities with no inter-connection, or dissolution cavities which appear to a limited extent near the sides of the valley. Since the current findings are based on test pitting and surface geological observations with no drilling or geophysical investigations conducted during this study, detailed geophysical studies and bore holes are necessary to understand better the conditions of subsurface cavities and sinkholes in the present study area.

4. Conclusions

The geotechnical and geological investigation ensures the safe and sustainable design of the Bargun dam, a vital infrastructure to address the region's acute water scarcity and socio-economic challenges. By applying diverse empirical, laboratory, and field investigation methods, significant variability in geotechnical parameters are shown, bearing capacity ranging from 0.55 to 23.22 MPa and permeability from 2.14×10^{-2} to 3.7×10^{-5} cm/s. Utilizing multiple assessment techniques shall enable to capture a comprehensive range of possible outcomes, enhancing the safety and reliability of foundation designs and

underscoring the importance of multi-method approaches in data-scarce environments.

Kinematic slope stability analysis showed both abutments are stable against planar and wedge failures; however, the left abutment is susceptible to oblique toppling, necessitating slope modification. The site is characterized by variable soil and rock conditions, with unsuitable foundation materials in some areas due to high plasticity and permeability. At the same time, some zones exhibit competent rock masses with acceptable strength and durability for dam construction. To ensure safety, it is advisable to base the dam on the bedrock by removing the shallow sediments from under the dam's base, since its low thickness will not present any particular difficulties.

Overall, the study provides critical insights into assessing stability and stress capacity in the Main-Gypsum-Formations, facilitating informed decision-making for future projects in similar geological contexts. The permeability and joint characteristics of the gypsum rock masses, particularly at the right abutment, suggest the need for targeted grouting to mitigate leakage risks. The seismic hazard is low, and the site meets the structural and material requirements for dam construction, providing implementation of appropriate engineering measures to address localized geotechnical challenges.

The current findings are based on test pitting and surface geological observations with no drilling or geophysical investigations conducted during this study. Therefore, detailed geophysical studies and bore holes are necessary to understand better the conditions of subsurface cavities and sinkholes in the dam foundation and reservoir area.

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