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GEOLOGICAL AND GEOTECHNICAL INVESTIGATIONS OF GALI ZAKHO TUNNEL, DUHOK, KURDISTAN REGION – IRAQ

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ABSTRACT

Geographically, northern Iraq, specifically on the border between Iraq, Turkey, and Iran, is considered a complex geological area consisting of different types of rocks, including sedimentary, igneous, and metamorphic. The height of the mountains in these areas ranges between 2,000 and 4,000 meters. Recently, many changes occurred in the region as a result of natural disasters such as earthquakes, random drilling of wells, and the rapid development of multi-story buildings, which led to many modifications in the geology of the Kurdistan Region. In order to construct any tunnel, the geological plan and profiles should be formed by evaluating the site investigations, and the boreholes should be drilled through the tunnel route. Accurate investigations about the geological structure and nature of the ground can be conducted during the excavation. Also, essential laboratory tests are done on core samples extracted from boreholes along the tunnel. The behaviors of the geological and geotechnical situation of the tunnel are presented, focused on the inlet of the tunnel. This work relied on previous geological investigation surveys that were conducted in the region and related to this subject, the results of geological field mapping and laboratory tests, and the geological and geotechnical parameters used for the design and construction of the tunnel. The best excavation method that can be used in this area is the New Austrian Tunneling Method (NATM), based on the economic and area conditions.

KEYWORDS: Tunnel; displacements; rock mass; excavation; Geotechnical and Geological condition

1. INTRODUCTION

Numerous Numerous tunnels have been excavated and constructed in a vary of geological and geotechnical stipulations in the Kurdistan Region. A whole identification of the rock mass ought to be made in the initial stage in order to actually design and effectively construct an underground tunnel. The conduct of the tunnel contour and the procedures for excavation, lining, settlement, and sooner or later safety management and monitoring are greatly influenced by the properties and structure of the rock mass. In Iraq, tunnel construction has a long history dating back to the Babylonian and Sumerian eras, etc. At the time, tunneling involved lighting a fire and using drills and saws to break up rocks. According to geological investigations carried out in the area, it has been determined that the geological structure in the region is made up of a range of soil types, inclusive of Silty and Sandy levels, sandstone, siltstone, claystone, both sparitic and clayey limestone, and Marl. Iraq was originally beyond the seismic zone, according to earlier tectonic investigations. Today, it is one of the seismically active regions and is especially energetic in the northern and eastern parts, which are consider a

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portion of the Arabian plate. The Iraqi standards give the full definition of the abbreviation (SORB), the American Association of Highway and Transportation Officials (AASHTO), the Q-system, the Rock Mass Rating (RMR) system, and the New Austrian Tunnel Method (NATM) were used in the design and construction of the Gali-Zakho tunnel.

The Gali-Zakho Tunnel, which is situated on the roadway that connects the borders of Kurdistan-Iraq and Turkey, was the first underground structure in Duhok City, on the route connecting Zakho-Duhok and Erbil. The tunnel consists of two individual tubes with a total length of 3.604 km (Mohammed, 2016). To categorize the rock masses and tunnel stability and support estimates. Prior researchers employed a variety of on-site and laboratory test methods, including the Q-system, Rock Quality Designation (RQD), joint conditions, stress reduction factor (SRF), Rock Mass Rating (RMR), and uniaxial compressive strength (UCS), see for example Palmstrom and Broch (2006), KOMURLU and DEMIR (2019), Antoni and Krzysztof (2022), Hoek and Diederichs (2006), Tajduś and Tajduś (2022), Dybeł et al. (2021), Barton et al. (1974), Dybe et al. (2021), Jun et al. (2021), and Singh and Coel (1999). To confirm and identify the topographical nature and indicate the geological structure of Iraq, in particular the northern region, many researchers and geologists conducted numerous experiments and in-situ testing on soil and rocks to specify the physical and chemical properties and their have an impact on the underground structures, specifically the tunnels due to their direct relationship with soil and rock mass. Researchers with experience in this area are: Abdulnaby (2018), DOSKI (2017), Karim and Al-Bidry (2020), AL-MOADHEN (2009), Fouad (2014), Sissakian et al. (2018), Garzic et al. (2019). Ebu Bekir (2020) investigated and evaluated the conditions of the NATM method on the Bolu tunnel; the results indicate that the solutions to the problems encountered in this tunnel are in conformity with the NATM principles. One of the important points in the design is knowing the characteristics and describing the rock mass accurately to describe the important parameters to ensure the stability of the rock mass. These parameters are generally as follows:

1. The compressive strength and modulus of elasticity of rock mass.
2. The rock quality designation (RQD)
3. Description of joints including direction, spacing and other conditions.
4. Description faults and folds.
5. In-situ stresses.
6. Water level and weathering and so on

2. Gali Zakho Tunnel - Case Study

2.1 Tunnel Description, Geological and Geotechnical Conditions

2.1.1 *Tunnel Description*

In the current work, the geotechnical and geological evaluations at the inlet section (Ch: 28+955-Ch: 28+967) of the Gali Zakho tunnel, Duhok-Zakho Highway, Duhok province, are described. For the purposes of tunnel design, the four boreholes (BH-1, BH-2, BH-3, and BH-4) dug at the inlet are evaluated at this section. The tunnel is made up of two parallel tubes, each measuring 110 m^2 in excavation area and 3.5 km in total length. Conventional and mechanized methods, including drilling and blasting, were used for tunneling. Generally, in any project, several geomechanically problems may occur during and after excavation. Therefore, it should be investigated carefully, such as for an expected large convergence in squeezing rock masses, settlement, spalling, rock-bursting, etc. The result of the geotechnical test was presented in detail using boreholes that were selected along the path on both sides of the inlet. The inlet elevation of the tunnel is approximately 651 m, located at Ch: 28 + 955.

2.1.2 *Geological and Geotechnical Evaluation at the Inlet of the Tunnel*

According to the data obtained from many boreholes, including: BH-1 at Ch: 28 + 986 (Left: 2.80 m.); BH-2 at Ch: 28 + 986 (Right: 29.50 m.); BH-3 at Ch: 28 + 938 (Left: 13.00 m.); and BH-4 at Ch: 29 + 008 (Right: 19.00 m.); Borehole locations BH-4A, BH-6A, BH-6D, and BH-9A at the tunnel route (between Ch: 29 + 095 and 32 + 535). According to the borehole (BH-1), the investigation hole is presented within the RQD values through the whole borehole, as shown in the table 1 below.

Table (1) :-Shows the outcome of borehole BH – 1 within the RQD values.

Sample	RQD%	Sample	RQD%	Sample	RQD%	Sample	RQD%
0.00 – 1.50	0	10.00 – 11.00	27	20.00 – 21.00	60	30.07 – 31.50	0
1.50 – 2.00	0	11.00 – 12.00	15	21.00 – 22.00	40	31.50 – 32.10	0
2.25 – 3.00	0	12.00 – 13.00	20	22.00 -23.00	10	32.10 – 33.10	0
3.05 – 4.00	0	13.00 – 14.00	25	23.00 – 24.00	36	33.10 – 34.10	0
4.00 – 5.00	0	14.00 – 15.00	20	24.00 – 25.00	10	34.10 - 35.10	0
5.00 – 6.00	0	15.00 – 16.00	0	25.00 – 26.00	0	35.13 – 35.87	0
6.00 – 7.00	0	16.00 – 17.00	0	26.05 – 27.00	0	35.87 – 36.10	0
7.00 – 8.00	0	17.00 – 18.00	0	2700 – 28.00	0	36.10 – 37.00	0
8.00 – 9.00	10	18.00 – 19.00	42	28.00 – 28.50	0	37.00 – 38.50	0
9.00 – 10.00	15	19.00 – 20.00	10	28.50 – 30.00	0	38.62 – 40.00	10

The borehole (BH – 2) log of the investigation is illustrated within the RQD values through the whole borehole as shown in table 2 below. The data obtained from boreholes BH-1 and BH-2 indicates the presence of two sets of discontinuity, groups of random joints. The sandstones exhibit a high degree of blockiness and contain joints that are partially closed, but mostly open at intervals ranging from 10 to 30 per cubic meter, with a joint interval greater than 5 mm. The sandstones are classified as medium stiff-soft, moderately weak, and moderately to highly weathered. The joint surfaces are rough or irregular, with planar defects, and generally of very poor quality, with hard clay filling. A single shear zone will be considered effective on competent rock at a depth less than 50 m if a discontinuity system is not observed at the subject area. The underground water level in both boreholes was approximately 13 m. Therefore, the dripping is expected to be predicted at 25 to 125 l/minute. Depending on the data resulted from borehole BH-3, the soil profile at this location is composed of 0.70 m of thick colluvial soil at the top, clay reaching up to the 9.50 depth under, sandstone between 9.50 and 19.50 m, and claystone afterwards. The soil profile starts from the ground surface at Ch: 28 + 986 (Left: 2,80 m). The BH-3 borehole log of the investigation is presented within the Rock quality designation (RQD) values through the whole borehole, which are shown in the table 3 below, and the average RQD value at the tunnel inlet portal is about (RQD = 16%).

Table (2) :-The outcome of borehole BH – 2 log within the RQD values.

Sample	RQD%	Sample	RQD%	Sample	RQD%	Sample	RQD%
0.00 – 0.70	0	10.00 – 11.00	10	21.00 – 22.00	50	31.50 – 32.10	0
0.70 -1.50	-	11.00 – 12.00	25	22.00 – 23.50	10	31.50 – 32.50	0
1.50 – 3.00	20	12.00 – 13.00	0	23.50 -25.00	0	32.50 – 33.50	0
3.00 – 4.50	18	13.11 – 14.00	15	25.00 – 26.00	0	33.50 – 35.00	70
4.50 – 5.50	50	14.00 – 15.00	0	26.05 – 27.00	10	35.00 - 36.50	0
5.50 – 6.00	10	15.00 – 16.00	28	27.00 – 28.00	0	36.50 – 36.75	0
6.00 – 7.50	0	16.00 – 17.00	95	28.00 – 29.30	0	36.75 – 37.55	80
7.95 – 9.00	13	17.00 – 18.50	10	29.30 – 30.00	0	37.55 – 39.00	0
9.00 – 9.50	0	18.50 – 19.50	0	30.07 – 31.00	80	39.06 – 40.00	0
9.50 – 10.00	60	19.50 – 21.00	0	31.00 – 31.50	10	-	-

Table (3) :-The outcome of borehole BH – 3 within the RQD values.

Sample	RQD%	Sample	RQD%	Sample	RQD%	Sample	RQD%
0.00 – 1.50	0	9.45 – 10.95	15	17.10 – 18.00	45	25.00 – 26.30	50
1.93 – 3.00	0	10.95 – 12.10	0	18.00 – 19.50	22	26.50 – 27.50	70
3.29 – 4.50	0	12.10 – 13.50	46	19.50 – 21.00	0	27.50 – 28.50	13
4.95 – 6.00	0	13.50 – 15.00	45	21.00 – 22.50	0	28.50 – 30.00	40
6.25 – 7.50	0	15.00 – 16.50	14	22.50 – 24.00	20	-	-
7.62 – 9.00	10	16.50 – 17.10	18	24.00 – 25.00	40	-	-

2.1.2.1 Rock Quality Designation (RQD):

RQD considered as an important factor that creates the different geotechnical evaluations interested in the design. RQD value Estimation:

$$RQD = 115 - 3.3J_V \qquad (1)$$

where:

J_v: the number of discontinuities of 1 m^3 rock mass

$J_v = 1/S_1 + 1/S_2 + \dots 1/S_n$

S_n: "n" the interval for discontinuity system,

According to BH-1 and BH-2 boreholes, the weighted average of RQD value through tunnel route is; RQD = 11 % . Regarding the results of boreholes BH-1, BH-2 and BH-3, the Uniaxial Compressive Strength of intact rock material (UCS) tests are performed on the core samples, the average (UCS) is;

$$UCS = (14.75+15.45+10.68+12.39+25.12+5.91+22.56+17.6+9.12+25.69+24.28+20.72+16.12+20.3+19.43+19.24+25.39)/17=304.48/17\approx 18MPa \qquad (2)$$

Average modified point load strength index taken from laboratory results are as follows:

$$J_{S(50)} = \frac{0.33+0.6+0.83+0.34+1.03+0.96+0.78+1.35+1.3+0.5+0.21+0.32+0.37}{13} = 0.69 \ MPa \qquad (3)$$

Regards to previous authors, the modified index of core diameter and the compression strength ratio varies between (20 to 25).

$$q_u = I_{S(50)} x \ 20 = 0.69 \ x \ 20 = 13.8 \ MPa \qquad (4)$$

As a result, the average uniaxial compressive strength of rock units presents at the inlet portal can be accepted as: q_u = 15 MPa. In this case, the range for q_u is between (5 to 25) MPa.

Table (4) :-Rating for discontinuities orientations for tunnelling (starting at tunnel inlet portals). (Bieniawski, Z.T. (1989))

Strike Perpendicular to Tunnel Axis				Strike Perpendicular to Tunnel Axis		Irrespective of Strike, Dip
Drive with Dip		Drive Against Dip				
Dip 45° to 90°	Dip 20° to 45°	Dip 45° to 90°	Dip 20° to 45°	Dip 45° to 90°	Dip 20° to 45°	Between 0° to 20°
Very Favourable	Favourable	Fair		Very Unfavourable	Fair	Fair

Table (5) :-Rating adjustment for joint orientations.

Strike and Dip Orientations of Discontinuities		Very Favourable	Favourable	Fair	Unfavourable	Very Favourable
Rating	Tunnel	0	-2	-5	-10	-12

In case of beginning the excavation from the inlet portal;

$$Basic\ RMR\ Value = 2 + 3 + 8 + 1 + 0 + 3 + 2 + 3 + 0 = 22 \quad (5)$$

$$Modified\ RMR\ Value = 22 - 2 = 20 \quad (6)$$

Table (6) :-Rock mass classes (RMR) and corresponding design parameters and engineering properties.

RMR	100 - 81	80 - 61	60 - 41	40 - 21	Less than 20
Class No	I	II	III	IV	V
Description	Very Good	Good	Fair	Poor	Very Poor

Table (7) :-Rock mass classes and corresponding design parameters and engineering properties

Class No	I	II	III	IV	V
Average Standup Time	20 Years for 15m. span	1 Years for 10m. span	1 Weak for 5m. span	10 Hours for 2.5m. span	30 Minutes for 1m. span
Cohesion of Rock Mass (c)	> 400 kPa	300 - 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Internal Friction Angle of Rock Mass ($\emptyset^{\circ}$)	$> 45^{\circ}$	$35^{\circ} - 45^{\circ}$	$25^{\circ} - 35^{\circ}$	$15^{\circ} - 25^{\circ}$	$< 15^{\circ}$

$$Basic\ RMR\ Value = 2 + 3 + 8 + 2 + 0 + 3 + 0 + 1 + 0 = 19 \quad (7)$$

Laboratory test results were used to estimate parameters for Hoek & Brown's failure criterion, including GSI (Geological compression strength index), UCS (Uniaxial compressive strength of intact rock), Ei (Elasticity modules of intact rock), and mi (Material index). For the Sandstone-Claystone located 9. 0 m below the ground surface at the BH-3 borehole, the average uniaxial compressive strength (UCS) is 15 MPa.

Table (8) :-Guideline for excavation and support of 10 m span rock tunnels in accordance with the RMR system (After Bieniawski 1989) *.

Rock Mass Class	Excavation	Rock Bolts (20 mm, Dia., Fully Grouted)	Shotcrete	Steel Sets
V	Multiple drift 0.5 – 1.5m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematically bolts 5 – 6m.long. Spaced 1 – 1.5m, in crown and walls with wire mesh. Bolt invert.	150 – 200mm. in crown, 150mm. in sides and 50mm on face	Medium to heavy ribs spaced 0.75m with steel lagging and Forepoling is required. Close invert.

* Shape: horseshoe. Width: 10m. Vertical stress:25MPa. Construction: Drilling and

blasting.

The material index (mi) for a rock sample was selected as 7, which was determined from triaxial test results for the rock unit. Additionally, the elasticity modulus (Ei) for Sandstone at the BH-3 borehole was estimated to be 2800 MPa, while the disturbance factor (D) was selected as 0.7. One of the outcomes of the GSI is to quantify rock properties. To achieve this objective, two additional parameters are necessary: Structural Rating (SR) and Surface Condition Rating (SCR). These can be computed using the following calculations:

$$SCR = R_r + R_W + R_f \qquad (8)$$

where:

R_r: Roughness Rating (R_r = 3) and is slightly rough,

R_w: Weathering rating (R_w =1) is Highly weathered,

R_f: Infilling rating (R_f = 0) is Soft, thickness > 5mm,

$$\therefore\ SCR = 3 + 1 + 0 = 4 \qquad (9)$$

An important factor that used for modification of GIS is called Volumetric joint count, $\vec{J_V}$(discontinuity/m^3) which suggested by ISRM (1981). Block dimension definitions of Sandstone and Claystone observed in BH-3 borehole are in the category of "very small block Crushed" and $\vec{J_V}$ (joint/m3) :10-30 >30 unit/m3, in this case; J_V = 30

Structural Rating (SR);

$$SR = -17.5\ x\ lnJ_V + 79.8 = -17.5\ x\ ln30 + 79.8 \cong 20 \qquad (10)$$

2.1.2.2 *Evaluation of the deformation modulus of rock masses using RMR*

In order to calculate the deformation modulus of rock masses and RMR, Nicholson and Bieniawski (1990) derived the following relation:

$$\frac{E_m}{E_i} = \frac{1}{100}\left(0.0028MR^2 + 0.9e^{\left(\frac{RMR}{22.82}\right)}\right) \qquad (11)$$

If the basic value for RMR = 19 and the average elasticity modulus of intact rock is Ei = 2800 MPa then;

$$\frac{E_m}{2800} = \frac{1}{100}\left(0.0028(21)^2 + 0.9e^{\left(\frac{19}{22.82}\right)}\right) = 147MPa \qquad (12)$$

Based on Hoek & Diederichs (2005), the rock mass elastic modulus can be estimated from the below equation;

$$E_m = E_i\left(0.02 + \frac{1-\frac{D}{2}}{1+e^{[(60+15D-GSI)/11]}}\right) \qquad (13)$$

If the estimated GSI = 20, the average intact rock elasticity modulus Ei = 2800 MPa and the disturbance factor is D = 0,7, then;

$$E_m = 2800\left(0.02 + \frac{1-\frac{0.7}{2}}{1+e^{[(60+15(0.7)-20)/11]}}\right) = 74MPa \qquad (14)$$

Table (9) :-Geotechnical parameters for (19 m overburden) tunnel portal cut and cover slopes

Uniaxial compressive strength	Geological strength index	Material index	Disturbance factor	Elasticity modulus	Unit volume weight	Cohesion	Internal friction angle	Deformation modulus
(MPa)	GSI	Mi	D	MPa	kN/m3	c	ϕ	MPa
UCS				-	γ	kPa	(º)	Em
15	20	7	0.7	2800	26	41	22	74

Table (10) :-Geotechnical design parameters summary, Table for sandstone present at the tunnel inlet portal

UCS, Uniaxial compressive strength (MPa)			15
GSI, Geological Strength Index			20
mi, Material Index			7
Ei, Elasticity Modulus (MPa)			2800
D, Disturbance Modulus (MPa)			0.7
γ, Unit Volume Weight (kN/m3)			26
H, Overburden Thickness (m)			19
ϕ, Internal Friction Angle (º)		GSI	22
c, Cohesion (MPa)		GSI	41
Strength Parameters for Hoek-Brown Failure Criterion	GSI	mb	0.086
		s	9.22e-6
		a	0.544
Em, Deformation Modulus (MPa). (*) suggested	Nicholson & Bieniawski		147
	Hoek @ Diederichs		74*

Laboratory test results and failure criteria can be obtained through Hooke and Brown's method, which includes parameters such as GSI (geological compressive strength index), UCS (uniaxial compressive strength of intact rock), Ei (modulus of elasticity of intact rock), and mi (material index). Tables (4 -10) provides a summary of the results obtained from BH-1, BH-2, and BH-3 boreholes. Additionally, the average value of the Uniaxial Compressive Strength (UCS) of the core samples collected from these boreholes is considered appropriate;

$$UCS_{av.} = (14.75 + 15.45 + 10.68 + 12.39 + 25.12 + 5.91 + 22.56 + 17.6 + 9.1 + 25.69 + 24.28 + 20.72 + 16.12 + 20.03 + 19.43 + 19.24 + 25.39 + 20.16 + 20.14 + 23.35 + 19.18 + 23.60 + 4.77)/23 = 415.66/23 \cong 18MPa \qquad (15)$$

The average value which is considered as appropriate for Elasticity modulus is;

$$Ei_{av.} = (2953.8 + 4370.7 + 1990.2 + 2563.7 + 2983.1 + 1021 + 2874.2 + 2161.8 + 1362.4 + 2755.9 + 2537.8 + 1905.3 + 1120.7 + 1530.2 + 2265.3 + 1765.9 + 2953.1)/17 = 39115.10/17 \cong 2300MPa \qquad (16)$$

To understanding the rock properties at the inlet of the tunnel, the surface condition rating (SCR) parameters are required for this purpose.

$$Surface\ \ Condition\ \ Rating(SCR) = R_r + R_w + R_f \qquad (17)$$

where:

Rr: Roughness rating is slightly rough and is equal to 3

Rw: Weathering rating is moderately weathered, Rw = 3

Rf: Infilling rating is hard, thickness >5, Rf = 2

∴ Surface Condition Rating (SCR)= 3 + 3 +2 = 8

2.1.2.3 Q – system:

Regarding rockmass classification systems of Barton et al., 1974 and Barton, 2000. Q-system has been used and the rock mass rating related to the units those will be encountered during tunneling are calculated.

$$Q = \left(\frac{RQD}{J_n}\right)\left(\frac{J_r}{J_a}\right)\left(\frac{J_w}{SRF}\right) \qquad (18)$$

where:

RQD/J_n: rates the block size and

J_r/J_a: rates the inter-block shear strength.

Typical values of these parameters are shown in Table 3.4. The relations between Q and RMR can be proposed such as:

$$RMR \approx 9lnQ + 44 \qquad (19)$$

Barton’s Q-system first in 1974 in Norway considers the following six criterion:

1. RQD (Rock quality designation)

2. Number of joint clusters Jn

3. Joint roughness Jr

4. Weathering of joints Ja (joint alteration)

5. Joint water reduction factor Jw

6. Stress reduction factor SRF

2.1.2.4 Joints

a) Joint set number (J_n): The outcome of the boreholes BH-1, BH-2, and BH-3 log data indicates that the depth ranged between 0.70 and 1.50 m. Therefore, a significant discontinuity measurement cannot be observed because of the thick top soil existence above and the clayey level (especially in the BH-3 borehole, 9.50 m thick), and it is estimated that there will be two discontinuity sets plus random joint sets. The thicknesses of joints vary between 10 and 30 cm. The Joint set number (J_n) will be as follows: Two Joint sets plus random discontinuities J_n = 6, then the value suggested to be used in tunnel portals is (2,0 x 6 = 12).

b) Joint roughness number for critically oriented joint set (J_r): The joint roughness number is about 1.5, depending on the properties of the soil that is low in strength and highly weathered, such as Sandstones, siltstones, and claystones, and whether discontinuities are rough, irregular, or planar.

c) Joint alteration number for critically oriented joint set (J_a): Since the rock mass is characterized as loose or completely fractured rocks, Therefore, J_a = 6.

d) Joint water reduction factor (J_w): The average water pressure was predicted to be between 2.5 and 10 kg/cm^2. Because of the Groundwater table levels that were easily observed at BH-1, BH-2, and BH-3 boreholes, which vary between 6.00 and 13.50 m, Therefore, the Joint water reduction factor (J_w) will be as follows: J_w = 0,33.

e) Stress reduction factor (SRF): The stress reduction factor is about 5 at the tunnel inlet with a depth of excavation of 50 m and single-weakness zones containing clay or chemically disintegrated rock.

f) Excavation support ratio (ESR): The excavation support ratio is about one approximately.

Because the joint set numbers are less than 3, the support pressure at the crown will be calculated with the below-mentioned formula:

$$P_{roof} = \frac{2J_n^{0.5}}{3J_r} Q^{-1/3} (Grimstad\ \ and\ \ Barton, 1993) \qquad (20)$$

In the case of two joint sets plus random discontinuities at portals, Jn = 12, Jr =1,5 and if the calculated Q = 0,0151;

$$\square\ P_{roof} = \frac{2(12)^{0.5}}{3\ x\ 1.5} (0.0151)^{-1/3} = 6.223MPa \qquad (21)$$

The support pressure on the side wall (P_{wall}) with Q = 0,0151 is;

$Q_{wall} = Q$;

$$\square\ P_{roof} = P_{wall} = 6.223MPa \qquad (22)$$

3. DISCUSSION

In this study, the excavation site for the Gali Zakho tunnel inlet was evaluated geologically and geotechnically. The place picked is a key sector connecting the Iraqi-Turkish border, acting as an economic and strategic conduit for the area. The region has had traffic accidents and other problems as a result of the area's rugged mountainous environment. The study used on-site investigations to pinpoint different factors and parameters necessary for the design process. Based on the findings, the research took into account a number of variables, such as the Q-system, Rock Quality Designation (RQD), joint conditions, stress reduction factor (SRF), Rock Mass Rating (RMR), and uniaxial compressive strength (UCS). It is a well-known fact that the way rocks form and the environment around them influence the properties of rock formations in different geographic condition and weathering. While certain formations are seen as powerful, others are weak or frail. When a rock block's strength is judged to be weak or has a low durability index, pre-reinforcements, steel sets, or rock bolt anchors are used to prevent collapses during excavation. However, this technique is not required when the values for the rock mass strength are high. An illustration of this is the Darbandikhan tunnel in the Kurdistan Region of Iraq. No lining has been placed because of the rocks' extreme hardness and resistance. The Gali Zakho tunnel's geological and geotechnical assessments are the main subject of this study, with particular attention paid to the tunnel's inlet at sections Ch: 28+955-Ch: 28+967 and its path at Ch: 28+967-Ch: 32+535. Numerous boreholes were dug in numerous areas, including the tunnel inlet, which was considered for the tunnel design, to assure reliable data. The NATM excavation support class during the tunnel excavation was found to be C2 and B3 at the inlet based on empirical RMR and Q-System classifications as well as Austrian Standard ÖNORM B2203. Rocks that are stable, hard, and long-lasting are typically grouped into grades A1 - A2, which are more stable and permit full-face excavation. Rock types in the C1–C2 range are those that show partial instability or fragility. Throughout the excavation and construction phases, these rocks need diligent monitoring.

Table (11) :-Excavation and Support Class through Tunnel Inlet.

Chainage	Q-System	RMR	NATM Class
28+955 – 29+005	0.0151	20	C2

Table (12) :-The Parameters Specified on Site and Laboratory Outcomes at the Tunnel Inlet

Chainage	Unit Weight [γ], (kN/m3)	Uniaxial compressive Strength of Intact Rock Material, (UCS) (MPa)	Geological Strength Index (GSI)	Intact Rock Material, mi	Disturbance factor, D
28+955 – 29+005	26	18	29	9	0.3

Commentary of the information presented in the table 14 below is as follows:

1. The determination of bolt length, spacing, and steel set intervals will be based on local geological conditions encountered during tunnel driving at the site, classified as B3 Non-Inverted, B3 Inverted, and B3-A Non-Inverted.

2. If the excavation process encounters bottom heave as an issue, an inverted section comprising top heading, bench, and invert excavation phases may be utilized instead of a non-inverted section. The need for this approach will be determined by the consultant and the engineer on-site.

Table 13. Elasto-plastic Parameters of Tunnel Inlet

Chainage	Overburden (m)	NATM Excavation and Support Class	Hoek-Brown Criterion			Deformation Modulus (GPa)	Elasticity Modulus (MPa)
			mb	s	a		
28+955 – 29+005*	10-25	C2	0.456	0.0002	0.524	0.127	2437
* Between Ch:28+955 – 28+967 NATM excavation and support class: C2-A.							

Table(14):- Support System Summary of Tunnel inlet

Chainage	NATM Class	Rock Bolt (L*a*b): L= Bolt Length, a=Interval at section and b=Interval at advance	Shotcrete (C20/25)	Wire Mesh (Q221x221)	S-piling	Steel Sets	Advancement Length	Section Type
28+955-28+967	C2-A	6m x 1.0m x 0.5m	30cm	Double Layer	At crown 120° zone, 3,5", L=12m injected steel pipes (25cm spacing)	I200 0.50m intervals	Top heading 0.5-1.0m, Bench 1.5-2.0m and Invert 3.0m	Inverted

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4. CONCLUSION

The purpose of this study is to provide an assessment of the geological and geotechnical investigations conducted by the General Directorate of Roads, Bridges, Construction, and Housing: Duhok (owner), LMAK Construction and Contracting (contractor), and Mega Engineering and Consultancy Co. (consultant). The primary objective of laboratory and in-situ investigations is to achieve the best design, construction of high quality, and safety at a reasonable cost. The rock mass and tunnel support can be categorized using the Rock Mass Rating (RMR), the Q-system, and the Geological Strength Index (GSI). The soil and tunnel structure may be impacted by the tunnel's depth or the in-situ stresses of the rock mass, which could result in settlement and/or lining cracks. Regional geological characteristics differ, with mountainous areas typically being sturdier. Geological investigations, such as core drilling and soil tests, should be carried out to evaluate the status of joints and cracks in the rock mass alongside the tunnel course in order to guarantee secure tunnel construction. Drilling operations may come upon unforeseen phenomena despite thorough investigation and tests. On the other hand, hydrogeological data must be gathered and examined in order to prevent unexpected events. It is essential to concentrate on locating fault lines, fault zones, joints, rock bursts, compressing ground, discontinuities, and other geological and geotechnical problems during the excavation and tunnel construction process. In the event of unexpected problems occurring during excavation and construction, the layout must be reviewed or additional tests must be carried out, whether on site or in the laboratory. It is advised to use forepoling and/or increase the thickness of shotcrete if fault lines are found. The number of drifts, the length and spacing of the bolts, the steel sets, and the type of supports are all influenced by the excavation technique as well as geological and geotechnical conditions. The data acquired from boreholes BH-1 and BH-2 refers that the presence of two sets of discontinuity, groups of random joints. The sandstones show off an excessive degree of blockiness and contain joints that are in part closed, however in most cases open at intervals ranging from 10 to 30 per cubic meter, with a joint interval increased than five mm.

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