https://doi.org/10.26682/sjuod.2020.23.2.43

Journal of University of Duhok, Vol.23, No.2 (Pure and Eng. Sciences), Pp 523-534, 2020 (Special Issue)

DEFORMABILITY AND SHEAR STRENGTH PARAMETERS OF THE FOUNDATION OF A PROJECT ON TANJERO FORMATION IN IRAQ USING RMR AND GSI

YOUNIS M. ALI ALSHKANE[1]
College of Engineering, University of Sulaimani, Kurdistan Region-Iraq

(Accepted for Publication: December 8, 2020)

ABSTRACT

Determination of the shear strength and deformability parameters of weak rocks are very significant in order to design the foundation of buildings on it economically and safely. Tanjero formation consists of an alternation of sandstone and silty marlstone and it is considered as a weak rock layer which makes it difficult to determine precisely its bearing capacity. Understanding the behavior of this formation under loading is one of the most challenges in geotechnical engineering. The formation underlies most of the areas in Sulaimani city and significant projects such as Shari Daik and Barzayakni Sulaimni projects in Sulaimani city in Iraq. In this paper, the mentioned parameters of this formation were predicted and recommended for the foundation of a multi-story building at Shari Daik project in Sulaimani city in Iraq based on Rock Mass Rating (RMR) and Geological Strength Index (GSI) rock systems.

KEYWORDS: Weak rock; GSI; Rock Mass; RMR; Shear strength parameters and Deformability.

1. INTRODUCTION

Understanding the engineering properties and bearing capacity of weak rocks is a challenge to geotechnical engineers and geologists (Ahmed and Jamin, 2018; Ioanna et al., 2009). Weak rocks often form a great portion of the shallow stratums all over the world especially in Sulaimani region in Iraq. Since the foundation of many construction projects are built on weak rock, it is crucial to investigate the geotechnical properties of weak rocks such as Tanjero formation in Iraq. Tanjero formation consists of alternation of thin layers of sandstone and silty marlstone (Buday, 1980). The rock formation at Shari Daik project at the foundation of building A10 was chosen as a case study which is located in Sulaimani city, especially at the Dabashan District opposite the Chavi land park. Figure 1 shows the site plan of the project. The project is a multi-story building (23 stories) and is constructed as a reinforced concrete frame building.

This paper includes the results of geotechnical investigation for the foundation of the project so as to estimate the shear strength parameters and modulus of deformation using both rock classification systems: Rock Mass Rating (RMR) by Bieniawski, (1989) and Geological Strength Index (GSI) by Hoek (2002). The field work includes the rock mass investigation by taking rock cores from the foundation of the project and field study for measuring the orientation of joint sets of the rock mass under the building. In addition, it includes the results of laboratory testing, subsurface exploration, assessment of test results and Conclusions.

Younis.ali@univsul.edu.iq
[1]Corresponding author: Engineering College, Sulaimani University, Kurdistan Region, Iraq

2. METHODOLOGY

2.1 Field Work

The preliminary soil investigation was carried out by digging two boreholes. The Boreholes have been drilled to depth of 3m and it was difficult to get core rock samples because the marl layer was breakable and thin (layers thickness almost less than 10 cm). Therefore, prism rock samples of size (5cm x 5cm x10cm) were taken from the rock blocks of the outcrop during excavation of foundation. Figure 1 displays the site plan of the proposed project area with locations of the boreholes. The rotary drilling machine was used to dig the boreholes. It should be noted that the thickness of Tanjero formation at the study area is more than 500m (Karim and Surdashy, 2006).

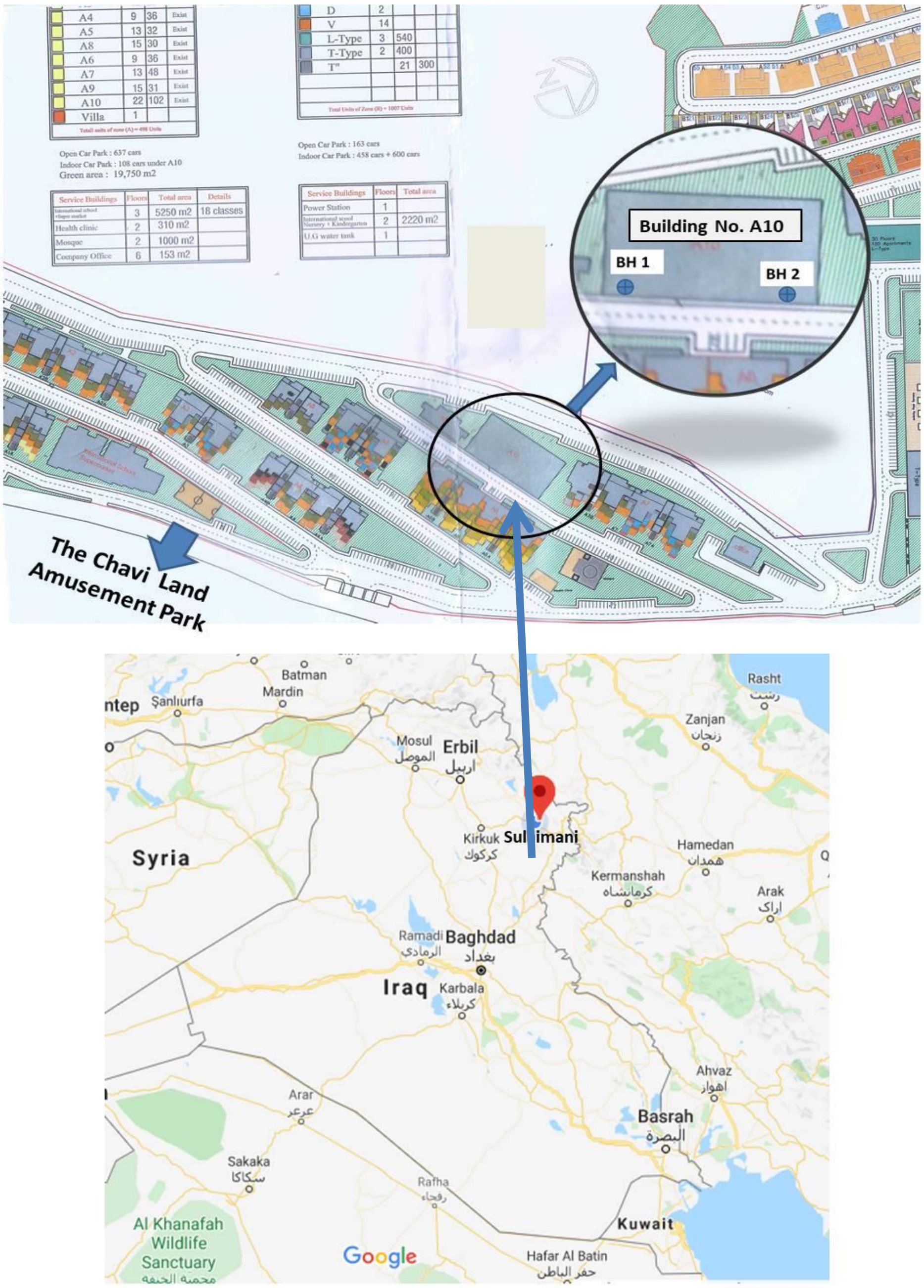

Fig. (1): Site plan of the project

Younis.ali@univsul.edu.iq
[1]Corresponding author: Engineering College, Sulaimani University, Kurdistan Region, Iraq

The foundation of the building is weak bed rock which consists of alternation of layers of marl and cemented sandstone. This bedrock is known as Tanjero Formation. Generally, there are a bedding layer and two joint sets. The dips and the dip directions of the planes are summarized in Table 1. Also, the planes are presented in Figure 2. The stereonet was drawn using softwatre GeoRose by Yong Technology Inc (2020).

Table (1): Dips and dip directions of the rock planes at Foundation of A10

Planes	Strike	Dip direction	Dip
Bedding	356	266	25±2.0
Joint set1	86	356	65±3.0
Joint set2	356	86	65±3.0

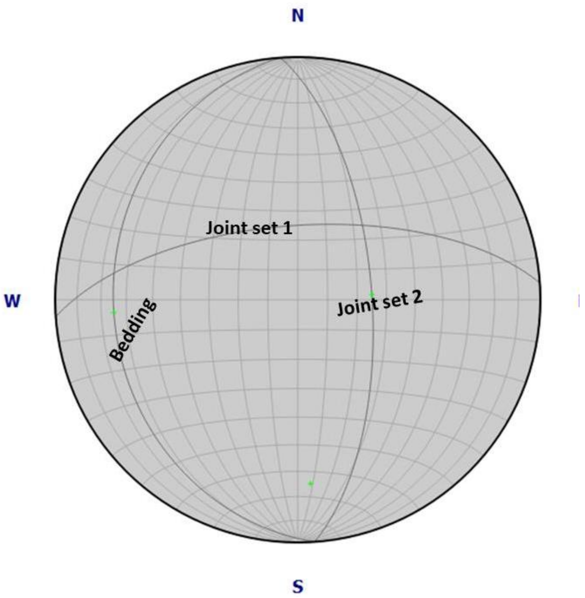

Fig. (2): Equal area equatorial stereonet.

2.2 Recovery of Rock Samples

Rock samples from un-weathered part of the foundation of the proposed building were generally obtained from the outcrops of the Tanjero formation at appropriate locations at the foundation level of the building. Six rock samples of size (5cm x 5cm x10cm) were obtained from both layers: Sandstone and marlstone.

2.3 Plate Load Test

The estimation of the modulus of subgrade reaction (ks) with allowable capacity of soils in situ of a foundation can be well estimated from the field load test. This test is usually referred to as the Plate Load Test. The test was done according to ASTM-1997 D1194. The plates are usually made of steel and are 150 mm to 762 mm in diameter and 25 mm thick (Das, 2007). In this work, the plate of a diameter of 300 mm which is manufactured by InfraTest Prüftechnik GmbH was chosen to conduct the test. The Plate Bearing Test Set 160 kN comprising (https://infratest.net/):

- measuring beam with ingle-gauge, made of Aluminium tubes with adjustable supports
- Plate size of Diameter of 300 mm with measuring tunnel.
- Set of pluggable extension rods.
- Upper ball and socket joint
- Hydraulic system 160 kN comprising piston.Th maximum pressure is about 1.1 MPa.
- The settlement is measured with a measuring bridge with gauge holder. Only one Dial gauge of 30 mm is used in this set

The plate size of 300mm was selected because the foundation of building A10 consists of thin layers of marlstone with sandstone which makes it reliable to use it. Figure 3 shows the plate load test which was performed in the field at depth of 2.0m from the lowest ground level in the front of building No. A10 at Shari Daik project. The excavator weight was used as a loading platform.

Younis.ali@univsul.edu.iq
[1]Corresponding author: Engineering College, Sulaimani University, Kurdistan Region, Iraq

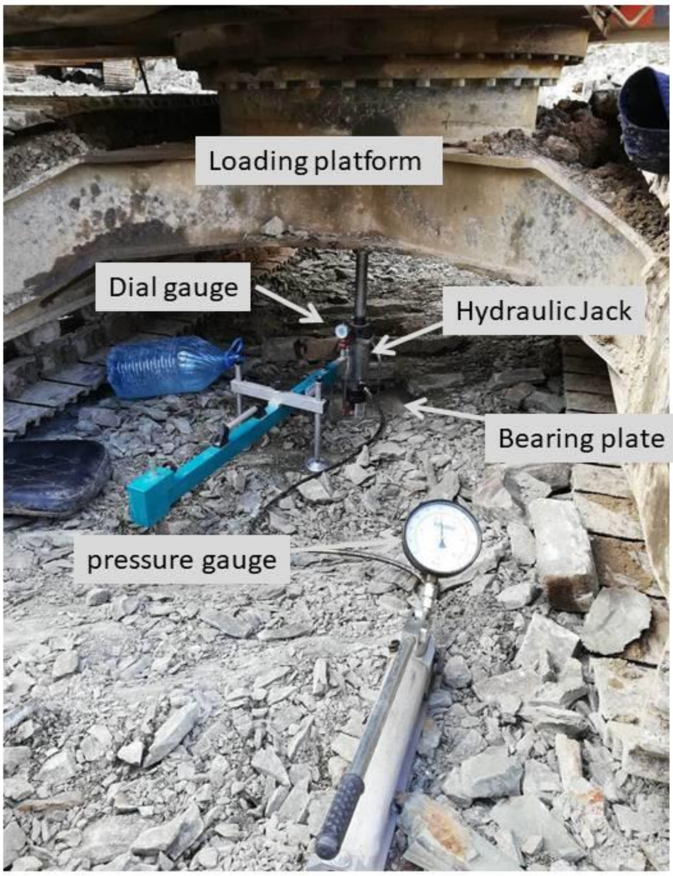

Fig. (3): Plate load test in the field

To conduct a test on a plate of diameter of 300 mm, it should be placed at the depth of the proposed foundation. The plate was loaded in 10 steps by means of a hydraulic jack. The maximum capacity of the load cell was about 1100 kPa. The settlement of the plate was measured for each load increment using the dial gauge. In this study, two points were tested. The tested locations were at the foundation center. The first point was tested under dry condition, whereas the second point was tested under saturated condition. Figure 4 displays the load settlement curve obtained from the tests, from which ks is determined. The ks was about 956938 kPa/m and 495050 kPa/m for dry and saturated condition, respectively. This indicates that the value of ks has been affected by saturation.

Younis.ali@univsul.edu.iq
[1]Corresponding author: Engineering College, Sulaimani University, Kurdistan Region, Iraq

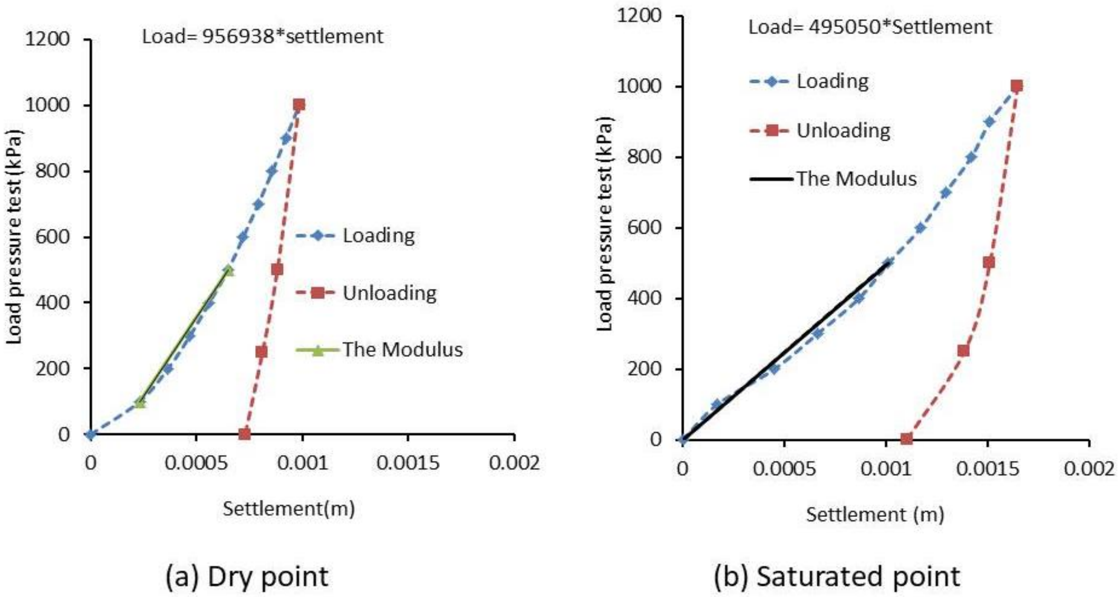

Fig. (4): Load settlement relationships (a) Dry point, (b) Saturated point

The value of subgrade reaction modulus is variable for a given soil and it depends on several factors, such as the length (L), width (B) of the foundation and the depth of foundation. "*A comprehensive study by Terzaghi (1955) of the parameters affecting the coefficient of subgrade reaction indicated that the value of the coefficient decreases with the width of the foundation* "(reported by Das, 2007). The value of ks is related to large square footing using Eq. 1 and Eq. 2 is used to covert the ks of square footing to ks of rectangular footing (Das, 2007)

$$ks(B,B) = k_{300mm} * \left(\frac{B+0.3}{2B}\right)^2 \quad (1)$$

$$ks(B,L) = ks(B,B) * \frac{\left(1+0.5\frac{B}{L}\right)}{1.5} \quad (2)$$

where: ks is the modulus of subgrade reaction of the rectangular foundation having dimension of B (35m) * L (58m) and k_{300mm} is the subgrade reaction modulus under a plate of diameter of 300mm. The value of ks from Equations (1 and 2) is 109.25 MN/m^3 for saturated point under foundation.

Also, the deformation modulus (Erm) was calculated from Figure 4 (b) for saturated case as **548.2** MPa.

3. RESULTS AND ROCK MASS ANALYSIS

3.1 Uniaxial Compression Strength (UCS)

The UCS was conducted for both core samples obtained from the foundation of building so as to use it to study the rock mass behavior. A strain rate of 0.002 mm/sec was used for application of axial load. The test was conducted as per ISRM (1981). The samples were saturated by putting them in water for three days.

The results of UCS of saturated samples are presented in Table (2). This property was used to study the rock mass behavior in the foundation of the proposed project. As can be seen that the UCS values of Marlstone is much less than that of Sandstone layer. Therefore, it is believed that the strength of Marlstone controls the behavior of rock mass at the foundation of Building A10. The average value of UCS of marl layer is 16.5 MPa. This value is used in the analysis of rock mass classification

Younis.ali@univsul.edu.iq
[1]Corresponding author: Engineering College, Sulaimani University, Kurdistan Region, Iraq

Table (2): UCS test results

Rock Sample No.	Sample type	Dry Density (gm/cm³	water content %	Uniaxial Compressive Strength (UCS) (MPa)
1	Sandstone	2.572	0.3	100.2
2	Sandstone	2.596	0.3	80.5
3	Sandstone	2.565	0.2	130.6
4	Marl	2.434	1.2	35.0
5	Marl	2.425	0.9	10.5
6	Marl	2.459	1.3	4.0

3.2 Chemical Tests

The chemical tests were conducted so as to find the sulfates content (SO_3 %) using XRF-Spectro and carbonates contents ($CaCo_3$) %) as percentages. These tests were conducted at Sulaimani Constructional laboratory (SCL). Also, organic material content was obtained. The results of chemical tests for the selected rock samples are presented in Table (3). According to these results, the ordinary Portland cement can be used in the foundations of the building.

Table (3): Chemical tests for selected soil samples

Sample type	SO3%	Organic materials.%	$CaCo_3$ %
Sandstone	0.014	1.38	51
Marlstone	0.108	1.66	27.5

3.3 RMR classification system

The Rock Mass Rating (RMR) system was used to classify the rock mass of building A10 foundation. The rock mass at the foundation was taken as one geotechnical unit. Because there are three planes (two joint sets with a bedding plane) as presented in Figure 2, the lowest rating should be taken into account. The lowest rating was for bedding layer which is the bedding plane that has a dip of about 27 degrees. To calculate the RMR, six parameters are required. From the first five parameters, the basic RMR can be calculated. Table 4 presents the parameters and their rating.

Table (4): Rating of the Building foundation

	Parameter	Values	Rating
	UCS (MPa)	16.5	2
	RQD %	25-50	8
	Measure from the chosen Scan Lines in Figure 5		
	Spacing (mm)	>60	8
Discontinuities condition	Discontinuity length (m)	>20m	0

Younis.ali@univsul.edu.iq
[1]Corresponding author: Engineering College, Sulaimani University, Kurdistan Region, Iraq

Aperture (mm)	1-5	1
Roughness	Slightly rough	3
Infilling	Soft filling <5mm	2
Weathering	slightly weathered	5
Ground water	Damp	10
Basic RMR		39

The basic RMR can be adjusted to find the final RMR using the joint orientations. The orientation is favorably (Dip=27 degree). According to Table 5, the final RMR is **37**.

The deformability and shear strength parameters of rock masses can be obtained using the final RMR which is 37. From section C of Table 5 the rock class number is IV, which indicates poor rock quality. By matching this number in section D of Table 5, the designed values for friction angle and cohesion were 23.4 degree and 184.2 kPa, respectively. The deformation modulus (E_{rm}) of rock masses was computed using Equation 3:by Serafim and Pereira (1983). As a result, the value of E_{rm} was calculated as 4.732 GPa.

$$E_{rm} = 10^{\frac{(RMR-10)}{40}} \text{ (in GPa)} \quad (3)$$

Younis.ali@univsul.edu.iq
[1]Corresponding author: Engineering College, Sulaimani University, Kurdistan Region, Iraq

Table (5): TheRMR_1989 classification system Table (Palmström, 2009)

A. Classification parameters and their ratings in the RMR system

	PARAMETER		Range of values // RATINGS						
1	Strength of intact rock material	Point-load strength index	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range, uniaxial compr. strength is preferred		
		Uniaxial compressive strength	> 250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	RATING		15	12	7	4	2	1	0
2	Drill core quality RQD		90 - 100%	75 - 90%	50 - 75%	25 - 50%	< 25%		
	RATING		20	17	13	8	5		
3	Spacing of discontinuities		> 2 m	0.6 - 2 m	200 - 600 mm	60 - 200 mm	< 60 mm		
	RATING		20	15	10	8	5		
4	Condition of discon-tinuities	a. Length, persistence	< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 20 m		
		Rating	6	4	2	1	0		
		b. Separation	none	< 0.1 mm	0.1 - 1 mm	1 - 5 mm	> 5 mm		
		Rating	6	5	4	1	0		
		c. Roughness	very rough	rough	slightly rough	smooth	slickensided		
		Rating	6	5	3	1	0		
		d. Infilling (gouge)	none	Hard filling		Soft filling			
			-	< 5 mm	> 5 mm	< 5 mm	> 5 mm		
		Rating	6	4	2	2	0		
		e. Weathering	unweathered	slightly w.	moderately w.	highly w.	decomposed		
		Rating	6	5	3	1	0		
5	Ground water	Inflow per 10 m tunnel length	none	< 10 litres/min	10 - 25 litres/min	25 - 125 litres/min	> 125 litres /min		
		p_w / σ1	0	0 - 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5		
		General conditions	completely dry	damp	wet	dripping	flowing		
	RATING		15	10	7	4	0		

p_w = joint water pressure; σ1 = major principal stress

B. RMR rating adjustment for discontinuity orientations

		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
RATINGS	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. Rock mass classes determined from total RMR ratings

Rating	100 - 81	80 - 61	60 - 41	40 - 21	< 20
Class No.	I	II	III	IV	V
Description	VERY GOOD	GOOD	FAIR	POOR	VERY POOR

D. Meaning of ground classes

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of the rock mass	> 400 kPa	300 - 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Friction angle of the rock mass	< 45°	35 - 45°	25 - 35°	15 - 25°	< 15°

3.4 Geological Strength Index (GSI)

Hoek (1994) and Hoek et al. (1995) developed GSI system as a direct replacement for Rock Mass Rating that was developed by Bieniawski and Orr (1976), and Bieniawski (1989). The GSI was suggested to develop a system to characterize the jointed rock mass and to estimate the strength and deformability indirectly (Cai et al., 2004). In this system, the parameters ($\emptyset$ and c) of the equivalent Mohr-Coulomb criterion as well as elastic deformation (E_{rm}) can be obtained. However, the Hoek-Brown criterion should only be used for isotropic rock masses (Fortsakis et al., 2012; Hoek and Brown, 1980).

Younis.ali@univsul.edu.iq
[1]Corresponding author: Engineering College, Sulaimani University, Kurdistan Region, Iraq

In this system, the shear strength parameter and deformation modulus of intact rock, found from laboratory tests, are reduced according to the properties of joint sets in the rock mass unit (Hoek and Brown, 2019). The texture of the rock mass is seen from the excavation of the rock mass of the investigated area (see Figure 5), which was revealed during the process of the foundation excavation of building no. A10 at the site. The rock mass is a fairly-interlocked blocky rock mass. This information can be used as input data to the software RocLab v 1.033 (Hoek et al.,2002) in order to get the deformability and strength properties of the foundation of the project. Figure (6) presents the suggested model for the studied area obtained from the RocLab software; the value of GSI is approximately equal to 34. The results of the mechanical properties are shown in Figure (7).

Hoek and Diederichs (2006) suggested two new relationships (Equations 4, 5) using a sigmoid function to predict the deformation modulus of a rock mass. These equations were developed after an analysis of field deformation moduli for a huge number of rock masses from Taiwan and China. Equation 4 is used where only GSI data are available, whereas Equation 5 can be used where real value of the intact rock modulus and GSI data are obtainable. These equations are generally applicable for isotropic rock mass.

$$E_{rm}(MPa) = 100{,}000\left(\frac{1-\frac{D}{2}}{e^{((75+25D-GSI)/11)}}\right) \quad (4)$$

$$E_{rm} = E_i\left(0.02+\frac{1-\frac{D}{2}}{e^{((60+15D-GSI)/11)}}\right) \quad (5)$$

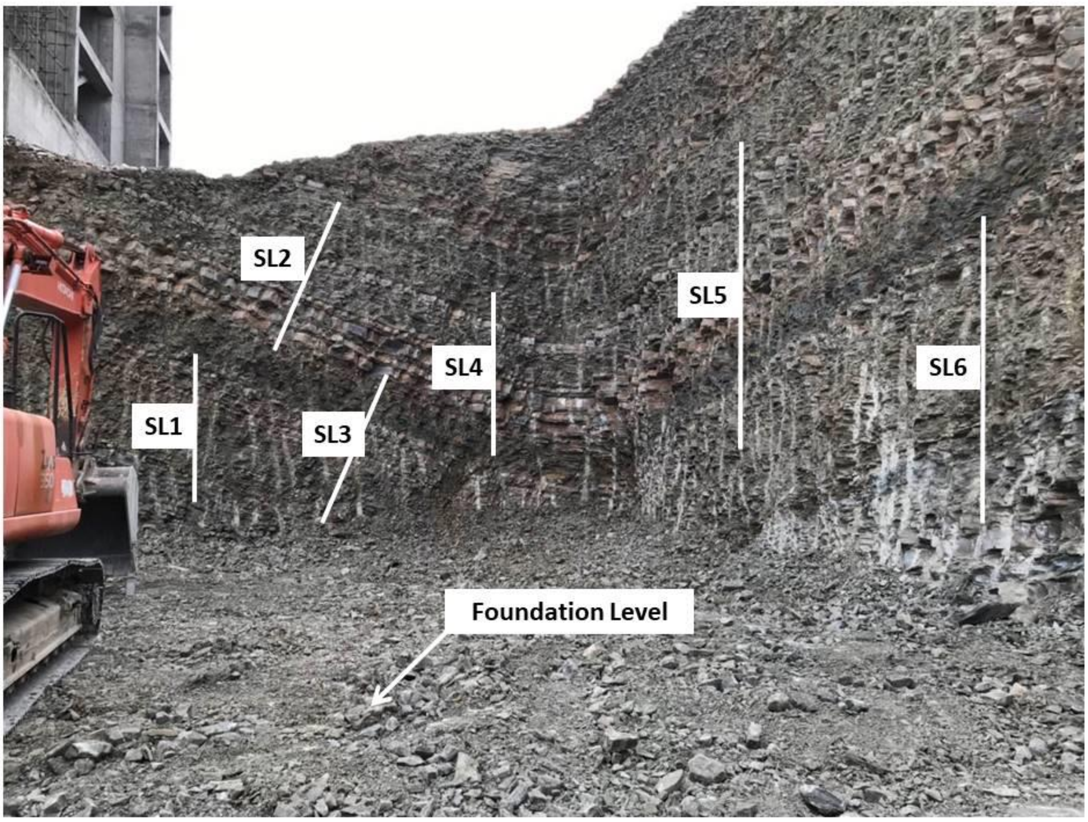

Fig. (5): the fabric of the rock mass at foundation of building A10. Note: SL indicates the Scan lines

Younis.ali@univsul.edu.iq
[1]Corresponding author: Engineering College, Sulaimani University, Kurdistan Region, Iraq

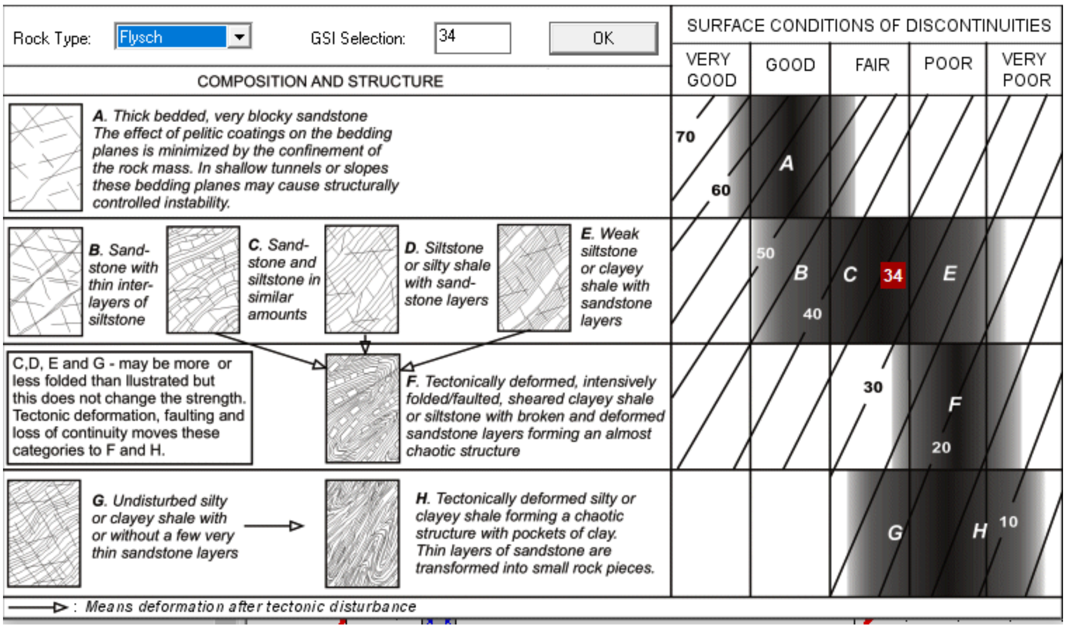

Figure 6 GSI value for the rock mass produced by RocLab code (Hoek et al., 2002)

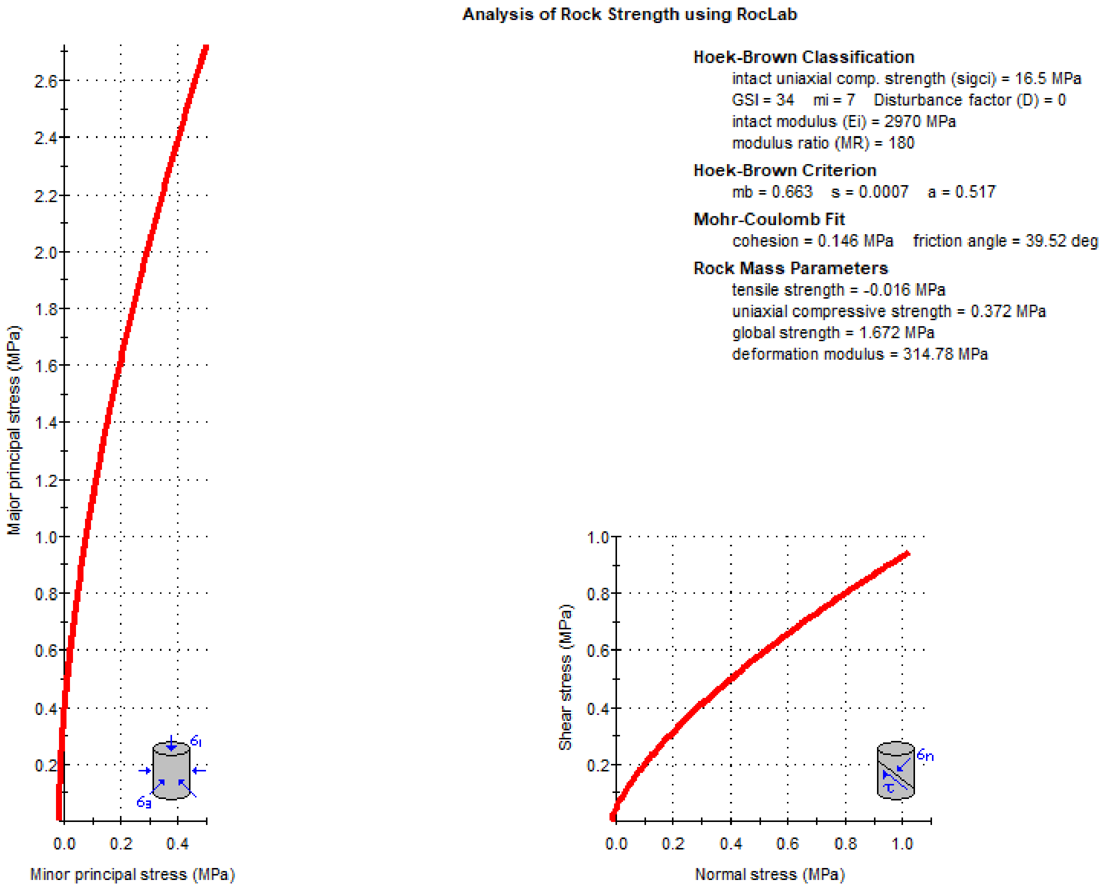

Fig. (7): Results of rock mass strength by RocLab code (Hoek et al., 2002)

4. DISCUSSION AND ASSESSMENT

The shear strength and deformability parameters produce by two methods are presented in Table 6. The friction angle predicted by RMR is less than that predicted by GSI, whereas the cohesion by GSI is less than that produced by RMR. It appears that GSI

Younis.ali@univsul.edu.iq
[1]Corresponding author: Engineering College, Sulaimani University, Kurdistan Region, Iraq

overestimates the friction parameter. This is also concluded by Alshkane (2015) when he used the Universal Distinct Element Code (UDEC) to analyse the shear strength parameters of a rock mass under a dam foundation.

Concerning the deformability properties, the result of Erm from plate load test was compared with the RMR and GSI results. It can be seen that the value of Erm produced by plate load test is near from the result of GSI than RMR. Therefore, the Erm by GSI is recommended for settlement calculation of the building foundation (A10). It was also concluded that plate load test can be certainly used to determine the deformability parameter of a rock mass, since there is a difficulty in testing large jointed rock samples in the laboratory at this time. The classification systems only provide the empirical equation to calculate the deformation modulus, and from Table 6 it can be concluded that the RMR overestimates the value of deformation modulus whereas GSI system gives reasonable result. Similar results were concluded by Alshkane (2015) when he used UDEC to analyze the foundation of Surqawshan dam in Sulaimani governorate in Iraq.

For quick prediction of deformability and strength parameters, especially for weak sedimentary rock like the Tanjero formation in Iraq, the deformation modulus can be estimated using GSI system; however, the shear strength parameters can be safely estimated using RMR. According to the results of this study, it can be suggested that this rock formation needs more investigation so as to design the foundation of building economically and safely.

Table (6): Strength and deformability parameters produce by different methods

Method	Index	Friction angle	Cohesion (kPa)	Elastic modulus (MPa)
RMR	37	23.4	184.2	4731.5
GSI	34	39.52	146.0	314.8
Plate Load Test				548.2

5. CONCLUSIONS

In this paper, the shear strength parameters and deformability were investigated using RMR and GSI rock systems for a real foundation of multistory building on a weak rock formation as a result the following conclusions are drawn:

1) The internal friction angle predicted by GSI is much higher than that predicted by RMR.
2) The equivalent cohesion of the rock mass produced by RMR is slightly higher than that by GSI system
3) The predicted modulus of deformability of the rock mass by GSI system is more realistic than that produced by RMR as compared with deformation modulus by field plate load test.
4) Based on load-settlement relationship under saturated condition, the modulus of subgrade reaction (ks) of 107.5 MN/m^3 can be considered in designing of the foundation.

Based on this study, the deformation modulus can be predicted from the GSI system, whereas the strength parameters can be predicted using RMR.

ACKNOWLEDGEMENTS

The research for this study was financially supported by South Kurdistan Company. Special thanks must to go to to Engineer Rozhgar Kamal Muhammad at Slemani Constructional Laboratory (SCL) in Slemani city in Iraq for conducting the plate load test. Also, I would like to acknowledge Geologist Washyar Ahmed Faraj at SCL and Engineer Shadman Shahab at Shari Daik project for their encouragement and support during this study.

Younis.ali@univsul.edu.iq
[1]Corresponding author: Engineering College, Sulaimani University, Kurdistan Region, Iraq

REFERENCES

Alshkane, Y. M. A. (2015). Numerical modelling investigation of rock mass behaviour under gravity dams (Doctoral dissertation, University of Nottingham).

Ahmad, N.R. and Jamin, N.H., 2018, April. Preliminary view of geotechnical properties of soft rocks of Semanggol formation at Pokok Sena, Kedah. In IOP Conference Series: Earth and Environmental Science (Vol. 140, No. 1, p. 012117). IOP Publishing.

Buday, T., 1980. The regional geology of Iraq: tectonism, magmatism and metamorphism (Vol. 2). State Organization for Minerals, Directorate General for Geological Survey and Mineral Investigations.

Bieniawski, Z. T. & Orr, C. M. 1976. Rapid site appraisal for dam foundations by the geomechanics classification. Proc. 12th Congr. Large Dams, ICOLD, Mexico City. ICOLD, 483-501.

Bieniawski, Z. T. 1989. Engineering rock mass classifications. NewYork, John Wiley & Sons.

Das, Braja , 2007 Principles of Foundation Engineering, Cengage Learning , SI Seventh Edition

Cai, M., Kaiser, P. K., Uno, H., Tasaka, Y. & Minami, M. 2004. Estimation of rock mass deformation modulus and strength of jointed hard rock masses using the GSI system. International Journal of Rock Mechanics and Mining Sciences, 41(1): 3-19.

Fortsakis, P., Nikas, K., Marinos, V. & Marinos, P. 2012. Anisotropic behaviour of stratified rock masses in tunnelling. Engineering Geology, 141–142(74-83.

Serafim, J. L. & Pereira, J. P. 1983. Considerations on the geomechanical classification of Bieniawski. Lisbon, Portugal: Proc. Int. Symp on Eng. Geol. And Underground Construction.

Hoek, E. & Brown, E. T. 1980. Empirical strength criterion for rock masses. J. Geotech. Engng Div., ASCE, 106(GT9): 1013-1035.

Hoek, E. 1994. Strength of rock and rock masses. *ISRM News Journal,* 2(2): 4-16.

Hoek, E., Kaiser, P. & Bawden, W. 1995. Support of underground excavations in hard rock. Balkema, Rotterdam.

Hoek, E. & Brown, E. T. 1997. Practical estimates of rock mass strength. International Journal of Rock Mechanics and Mining Sciences, 34(8): 1165-1186.

Hoek, E., Carranza-Torres, C. & Corkum, B. 2002. Hoek-Brown failure criterion-2002 edition. In: Proceedings of NARMS-TAC Conference, Toronto, Canada. 267-273.

Hoek, E. & Diederichs, M. S. 2006. Empirical estimation of rock mass modulus. International Journal of Rock Mechanics and Mining Sciences, 43(2): 203-215.

Hoek, E. and Bray, J. W. 1981. Rock Slope Engineering Inst. Of Mining and Metallurgy. London. 358 p.

Hoek, E., & Brown, E. T. (2019). The Hoek–Brown failure criterion and GSI–2018 edition. Journal of Rock Mechanics and Geotechnical Engineering, 11(3), 445-463.

Palmström, A. 2009. Combining the RMR, Q, and RMi classification systems. *Tunnelling and Underground Space Technology,* 24(4): 491-492.

Ioanna, I., Dimitrios, R., Theodora, P. and Paris, T., 2009. Geotechnical and mineralogical properties of weak rocks from Central Greece. *Open Geosciences*, *1*(4), pp.431-442.

ISRM 1981. Rock Characterization, Testing and monitoring. ISRM Suggested Methods. Pergamon Press.

Karim, K. H., & Surdashy, A. M. (2006). Sequence stratigraphy of Upper Cretaceous Tanjero Formation in Sulaimaniya area, NE-Iraq. Kurdistan Acad. J, 4(1), 19-43.

Yong Technology Inc (2020). GeoRose 0.5. 1–Rose Plot Software. Edmonton, Canada {http://www.yongtechnology.com/download/georose}

Younis.ali@univsul.edu.iq

[1]Corresponding author: Engineering College, Sulaimani University, Kurdistan Region, Iraq