Journal of Geotechnical Geology

Zahedan Branch,
Islamic Azad University

Journal homepage: geotech.iauzah.ac.ir

Application of the Q-slope classification system for slope stability assessment of the south flank of the Assalouyeh anticline, South Pars Zone

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ARTICLE INFORMATION

Received 22 January 2018
Revised 03 March 2018
Accepted 11 April 2018

KEYWORDS
Rock slope engineering; Geomechanics; Q-slope system; Geo-hazard; South Pars Zone.

ABSTRACT

Application of empirical rock-slope engineering classifications to the stability assessment of rock slopes is considered a basic rule that is obeyed by most of the geological engineers around the world. Some of these classification methods that are applied for special design and assessment in geo-structures such as: slope stability, mining activities, excavation, road/railway cutting, etc. are effectively used for reinforcement. The Q-slope classification system which is developed to describe is continuous rock-slope conditions is used for engineering judgment regarding slope stability. This work is focused on the Assalouyeh anticline's south side which is located in the South Pars Zone (SPZ) in southern Iran. According to the results of the study on 55slopes in the SPZ and implementation of the Q-slope system, a major part of the slopes in terms of sustainability possess critical/uncertain conditions (25 cases), 10 slopes are considered as unstable and 20slopes are classified as stable.

1. Introduction

Empirical methods in geotechnical assessment of rock structures are utilised based on the classification systems which are developed for stability analyses and engineering design. Rock mass classification systems by collection, ranking and quantification of information are related to the geological conditions, geometrical properties, discontinuity network, seepage, etc. Azarafza et al. (2013; 2017a) attempted to estimate the design parameters such as resistivity, deformability, in-situ stress field and/or reliability since these design parameters are considered to be a very important part of geotechnical engineering design. Although Ritter (1879) is the first person who used the primitive rock classification in tunnel designs, Terzaghi (1946) is the first scientist who proposed the rock mass classification system for steel frame tunnel support design. He presented a factor named 'rock load factor (Hp)' and defined it as the tunnel roof loosening zone height which is likely to load the steel arches or overburden materials that are classified in seven groups from good (I) to bad (IX) conditions and decrypted as hard and intact group (I) to swelling rock group (IX). This classification system is known as Terzaghi's rock-load theory. Lauffer (1958), as based on the rock-load theory presented the stand-up time related rock mass classification for unsupported tunnels to suggest support systems. Deere et al. (1966, 1970) modified Terzaghi's theory by introducing the rock quality designation (RQD) as a measure of rock quality. These scholars distinguished between blasted and machine excavated tunnels and proposed guidelines for the selection of steel arches, rock bolts and shotcrete supports for 6-12m diameter tunnels in rock masses. Cecil (1970) finalised Terzaghi's system and provided qualitative information on rock mass properties, but some limitations were considered. Deere and Deere (1989) modified the RQD system that was used by different researchers such as Palmstrom (1996, 2005), Şen and Eissa (1991), Romana (1993), Hudson and

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Harrison (1997), Singh and Goel (2011) in regards to rock mass classification. Wickham et al. (1972) introduced the rock structure rating (RSR) system for a quantitative description of the rock mass quality and appropriate ground support (in particular steel-rib support) which was modified by Skinner (1988). Bieniawski (1973) on the basis of his experiences in shallow tunnels in sedimentary rocks developed the geomechanics classification/ rock mass rating (RMR) system which was modified from 1974 to 1989. The latest version of RMR is presented in 1989 (Bieniawski, 1989). Bieniawski in 1984 presented the rock mass excavatability index for TBM, (RMEI) which was utilised for TBM excavations in rock masses. Barton et al. (1974) originally proposed the Q-system (which is known as rock tunnelling quality index) at the Norwegian Geotechnical Institute (NGI) based on approximately 200 case histories of tunnels and caverns. Kumar (2002) reported that there are 1260 case records to prove the efficiency of this design approach and that it was the best classification system for tunnel supports. Barton and Grimstad (2014) have presented the latest modification and application of the Q-system to tunnelling and rock cavern related engineering cases. Franklin et al. (1972) have performed research on rock material strength and excavation (digging, ripping, and blasting). Singh et al. (1986, 1987) developed the rippability index classification (RIC) based on Franklin's research and applied their classification to a number of ripping cases in United Kingdom and Turkey. Abdullatif and Cruden (1983), by comparing the RMR, Q-system and Franklin classifications, stated that block size and rock strength were the main parameters of all classifications. Palmstrom (1995) proposed the rock mass index (RMi) for rock mass strength characterisation that was based on the uniaxial compressive strength (UCS) of the intact rock material and jointing condition. Chauhan (1982) proposed the 'rate of tunnelling' classification for the realistic assessment of the tunnel status which was based on ground/job conditions, and management factors for long tunnels (Singh and Goel, 2011). Hoek and Brown (1997) introduced the geological-empirical based method, named as the Geological Strength Index (GSI) and utilised it in both hard and weak rock masses for estimation of the rock mass strength and deformation modulus (Hoek, 2006). GSI modification and application was presented by Marinos and Hoek (2000), Cai et al. (2004), Marinos et al. (2005), and Hoek and Carter (2013).

The RMR and Q-system represent an enormous evolution in experimental rock engineering and rock structure geomechanical classification. Some of the most important categories in rock engineering based on RMR and Q systems are as follows:

- Rock Mass Strength, RMS (Stille et al., 1982),
- Modified Basic Rock Mass Rating, MBR (Kendorski et al., 1983),
- Modified-Mining Rock Mass Rating, M-MRMR (Haines and Terbrugge, 1991),
- Rock Mass Number, N (Goel et al., 1995a),
- Modified-Rock Mass Rating, M-RMR (Unal, 1996),
- Q_{TBM} (Barton, 1999),
- Rock Condition Rating, RCR (Goel et al., 1995b),
- Mining Rock Mass Rating, MRMR (Laubscher, 1977),
- Simplified Rock Mass Rating, SRMR (Brook and Dharmaratne, 1985),
- Slope Mass Rating, SMR (Romana et al., 2003),
- Chinese Slope Mass Rating, CSMR (Chen, 1995),
- Slope Stability Probability Classification, SSPC (Hack et al., 2003),
- Alternative Rock Mass Classification System (Pantelidis, 2010),
- Global Slope Performance Index, GSPI (Sullivan, 2013), and
- Q-slope (Bar and Barton, 2017).

Among these classifications, some have been developed for specific purposes like slope or cavern engineering design and some have been developed for general assessments (Azarafza et al., 2017b). SMR, CSMR, SSPC, and Q-slope are specifically designed for the empirical evaluation of rock slope stability. The Q-slope presented in 2016 is based on the Q-system to estimate rock slope stability (Bar and Barton, 2016). In the study presented herein, the Q-slope classification system is utilised to evaluate slope stability analyses for the South Pars Zone (SPZ) construction region that is located in Assalouyeh, southwest of Iran.

2. Methodology

The Q-slope system is developed from Barton's Q-system which was developed for underground stability analysis and support system design. This system was applied to reduce maintenance or bench-width requirements of slopes (natural cut and open-pit mining slopes) which allow geo-engineers to assess in-situ excavated rock slope stability, and make slope angle adjustments as rock mass conditions become evident during construction. Barton and Bar (2015) recommend that it be used for all type of rock slope failures such as planar, wedge, toppling and local debris failures. Consideration of geometrical features, strength conditions and the in-situ stress field (Nikoobakht and Azarafza, 2016) is an important advantage of this classification which is easily used in open-pit mining, slope geometrical stabilisations, slope cuttings, roadway/railway excavations, residential sites, hilly terrain areas, etc. It should be mentioned that the Q-slope classification system is not intended for large slope stability assessment and using the system for these cases must be with caution.

2.1. Standard Q-system

Based on Barton et al. (1974) experimental works on underground spaces at NGI, the Q-system which is known as rock mass quality or rock tunnelling quality index was introduced and is based on the following causative factors (Barton and Grimstad, 2014):

$$Q = \frac{RQD}{J_n} \frac{J_r}{J_a} \frac{J_w}{SRF} \tag{1}$$

Table 1 Calculation of RQD for surface investigations

No.	Relationship	Parameters	References
1	$RQD = 115 - 3.3 J_v$	J_v: Sum of the number of joints per unit volume	Palmstrom (1996)
2	$RQD = 110 - 2.5 J_v$		Palmstrom (2005)
3	$RQD = 100(\lambda t + 1) e^{-0.1\lambda}$	t: Conventional threshold value (0.1m) λ: discontinuity frequency	Hudson and Harrison (1997)
4	$RQD = (\frac{V_f}{V_l})^2 \cdot 100$	V_f: In-situ compressional wave velocity V_l: Compressional wave velocity in intact rock core	Singh and Goel (2011)
5	$RQD = 110.4 - 3.68\lambda$	Modified Palmstrom relation for RQD>50% and 6<λ<16	Romana (1993)
6	$RQD = A^x B^y \cdot D_v$	A^x: Coefficient (105 to 120) B^y: Coefficient (2 to 12) D_v: Palmstrom's J_v	Singh and Goel (2011)
7	$RQD = 100\left(1 + \frac{0.1 J_v}{1+\alpha+\beta}\right)\exp\left(-\frac{0.1 J_v}{1+\alpha+\beta}\right)$	J_v: Palmstrom's J_v A and β: Heterogeneity coefficient	Şen and Eissa (1991)

where, RQD is Deere's Rock Quality Designation, J_n is the number of joint sets, J_r is the joint set roughness, J_a is joint set alteration, J_w is joint water reduction factor, SRF is in-situ stresses reduction factor based on tunnelling conditions. In surface investigation, RQD is evaluated by some empirical formulations based on discontinuity network distribution as presented in Table 1. In the Q-system the three main elements (Eq. 1), which are approximate measures of block size (RQD/J_n),inter-block shear strength (J_r/J_a) and active stresses (J_w/SRF) are evaluated based on the field observations of the rock mass. The Q-value can vary from 0.001 (Exceptionally poor) to 1000 (Exceptionally good).

2.2. Q-slope

The Q-slope classification system consists of the same six parameters of the standard Q-system, namely, RQD, J_n, J_r, J_a, J_w, and SRF but for using the slope assessment they are modified as RQD, J_n, J_r, J_a, J_{wice} and SRF_{slope} (Barton and Bar, 2015). The Q-slope formulation for slope stability assessment is presented by Bar and Barton (2017) as:

$$Q_{slope} = \frac{RQD}{J_n}\left(\frac{J_r}{J_a}\right)_0 \frac{J_{wice}}{SRF_{slope}} \quad (2)$$

where, J_{wice} is environmental and geological condition number, SRF_{slope} is three strength reduction factors from SRF_a (physical condition number), SRF_b (stress and strength number), and SRF_c (major discontinuity number). The other factors remain unchanged from the Q-system (Barton and Bar, 2015). Also, Bar and Barton (2016) defined the O-factor which covers the J_r/J_a ratio as the orientation factor. In the Q-slope and Q-system, the three main elements are characterized as block size (RQD/J_n), inter-block shear strength (J_r/J_a) and active stress or external factors (J_{wice}/SRF_{slope}). In this case, the minimum shear strength of favourable is J_r/J_a and the average shear strength for wedges is $(J_r/J_a)_1 \times (J_r/J_a)_2$ which the shear resistance (τ) is approximately evaluated as (Bar and Barton, 2017):

$$\tau \approx \sigma_n \arctan\frac{J_r}{J_a} \quad (3)$$

These researchers have presented the values ranging for estimation of Q-value based on field observations of the rock mass in Tables 2 to 7, which describe all of the Q-slope parameter ratings (Barton and Bar, 2017).The introduced Q-slope stability chart has created an easier way for design and analyses as illustrated in Figure 1. As seen in this figure, the areas associated with uncertainties must be calculated carefully and deserve more accurate formulations.

Table 2 The RQD factor description (Barton and Bar, 2017)

Description*	RQD-value (%)	Class
Very poor	0-25	A
Poor	25-50	B
Fair	50-75	C
Good	75-90	D
Excellent	90-100	E

* A nominal value of 10 is used to evaluate Q-slope.

Table 3 The J_n factor description (Barton and Bar, 2017)

Description*	J_n-value (%)	Class
Massive	0.1-1	A
1 joint set	2	B
1 joint set+ random	3	C
2 joint set	4	D
2 joint set+ random	6	E
3 joint set	9	F
3 joint set+ random	12	G
Heavily jointed	15	H
Crushed rock	20	J

* The description is used for small-scale and intermediate-scale features, 1.0 is added when the joint set mean spacing is greater than 3m.

Table 4 The J_r factor description (Barton and Bar, 2017)

Description*	J_r-value	Class
Discontinuous joints	4	A
Rough or irregular, undulating	3	B
Smooth, undulating	2	C
Slickensided, undulating	1.5	D
Rough or irregular, planar	1.5	E
Smooth, planar	1.0	F
Slickensided, planar	0.5	G
Zone containing clay minerals thick enough to prevent rock-wall contact	1	H
Sandy, gravely or crushed zone thick enough to prevent rock-wall contact	1	J

* The A to G classes for rock-wall contact and contact after shearing, and H-J classes for no rock-wall contact for shearing condition.

Table 5 The J_a factor description (Barton and Bar, 2017)

Description*	J_a-value	Class
Tightly healed, hard, non-softening, impermeable infilling	0.75	A
Unaltered joint walls, surface staining only	1	B
Slightly altered joint walls. non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2	C
Silty/sandy clay coatings, small quantities of clay disintegrated rock, etc.	3	D
Softening/low friction clay mineral coatings, and small quantities of swelling clays	4	E
Sandy particles, clay-free disintegrated rock, etc.	4	F
Strongly over-consolidated non-softening clay mineral infillings	6	G
Medium/low over-consolidation, softening, clay mineral infillings	8	H
Swelling-clay infillings	8-12	J
Zones/bands of disintegrated or crushed rock and clay	8-12	M
Zones/bands of silty/sandy-clay, small clay fraction	5	N
Thick, continuous zones or bands of clay	12-20	OPR

* A to E classes for rock-wall contact (no clay infillings, only coatings), F to J classes for rock-wall contact after some shearing (thin clay infillings, probable thickness ≈1-5 mm), M to OPR classes for no rock-wall contact when sheared (thick clay/crushed rock infillings).

Table 6 Calculation of RQD for surface investigations

Description*	J_{wice}-value			
	Desert environment	Wet environment	Tropical storms	Ice wedging
Stable structure, competent rock	1	0.7	0.5	0.9
Stable structure, incompetent rock	0.7	0.6	0.3	0.5
Unstable structure, competent rock	0.8	0.5	0.1	0.3
Unstable structure, incompetent rock	0.5	0.3	0.05	0.2

* When drainage is installed: $J_{wice}\times1.5$, when reinforcement is installed: $J_{wice}\times1.3$, when drainage+ reinforcement are installed: $J_{wice}\times1.5\times1.3$.

Table 7 The SRF factor description (Barton and Bar, 2017)

Description	Value	Class
(a) SRF_a		
Slight loosening due to surface location, disturbance from blasting or excavation	2.5	A
Loose blocks, signs of tension cracks and joint shearing, susceptibility to weathering, severe disturbance from blasting	5	B
As B, but strong susceptibility to weathering	10	C
Slope is in advanced stage of erosion and loosening due to periodic erosion by water and/or ice-wedging effects	15	D
Residual slope with significant transport of material down slope	20	E
(b) SRF_b		
Moderate stress-strength range (σ_c/σ_{max}= 50-200)	1-2.5	F
High stress-strength range (σ_c/σ_{max}= 10-50)	2.5-5	G
Localized intact rock failure (σ_c/σ_{max}= 5-10)	5-10	H
Crushing or plastic yield (σ_c/σ_{max}= 2.5-5)	10-15	J
Plastic flow of strain softened material (σ_c/σ_{max}= 1-2.5)	15-20	K
(c) SRF_c		
Major discontinuity with little or no clay	1-8	L
Major discontinuity with RQD_{100} = 0 due to clay and crushed rock (RQD_{100}=1m perpendicular discontinuity sample)	2-16	M
Major discontinuity with RQD_{300} = 0 due to clay and crushed rock (RQD_{300}=3m perpendicular discontinuity sample)	4-24	N

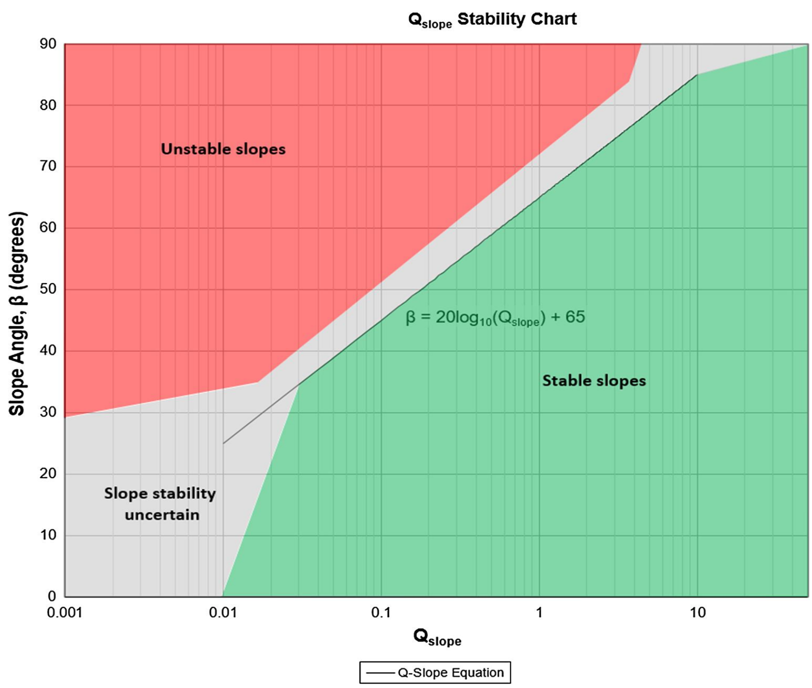

Figure. 1. The Q-slope chart of stability (Barton and Bar 2015; 2017)

3. Studied case

The South Pars Zone (SPZ) is located within the Assalouyeh region, Bushehr province in southwest of Iran and in the northern part of the Persian Gulf and southeast of the Assalouyeh anticline. It covers an area of approximately 100 square kilometres (Azarafza et al., 2014a) and mostly includes various complexes, facilities, plants and refineries known as ‘phases’ (Azarafza et al., 2014b). In terms of topography, SPZ is a narrow plain that is limited by the sea to the south and by mountains to the north. The location of the SPZ and its topography changes is shown in Fig. 2. In terms of geology, SPZ is mostly covered by alluvium deposits which are the result of erosion and sedimentation related to the formations of the area that include (Aghanabati, 2004):

- Hormoz series (Evaporate deposits
- Dehram group (Shale, carbonates, anhydrite, conglomerate, sandstone),
- Neyriz group (Thin bedded calcareous sandy shale, dolomite, marl, anhydrite),
- Dashtak group (Shales, dolomite, anhydrite),
- Khanehkat group (Massive dolomite)
- Khami group(Limestone, shale, dolomitic limestone, anhydrite, dolomite),
- Bangestan group (Limestone, shale),
- Pabedeh formation (Calcareous marl, shale, gray marl),
- Asmari formation (Limestone, dolomitic limestone),
- Gachsaran formation (Anhydrite, calcareous marl, salt, marl),
- Guri formation (Limestone),
- Mishan formation (Gray and creamy marl),
- Aghajari formation (Marlstone, limestone and sandstone),
- Bakhtiari formation (Conglomerate, sandy marly conglomerate),
- Alluvium (Unconsolidated soil, clay, sand, gravel).

Figure 3 shows the geological map of SPZ. As seen in Figs. 2 and 3, the main part of the rocky outcrops is located in the north side of SPZ and is associated with many localised rock slope instabilities. These failures occurred during the slope cutting for constructions (especially road, corridors, and channels) and excavations. In order to achieve an appropriate design, the stability analysis of these slopes during men-activities is necessary. For this purpose, the Q-slope system is used for stability investigation of the slopes as a primary stabilisation step

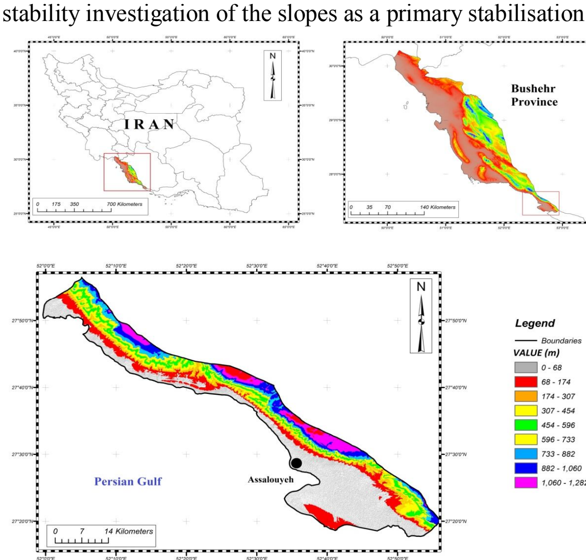

Figure. 2. SPZ and topography changes from DEM data

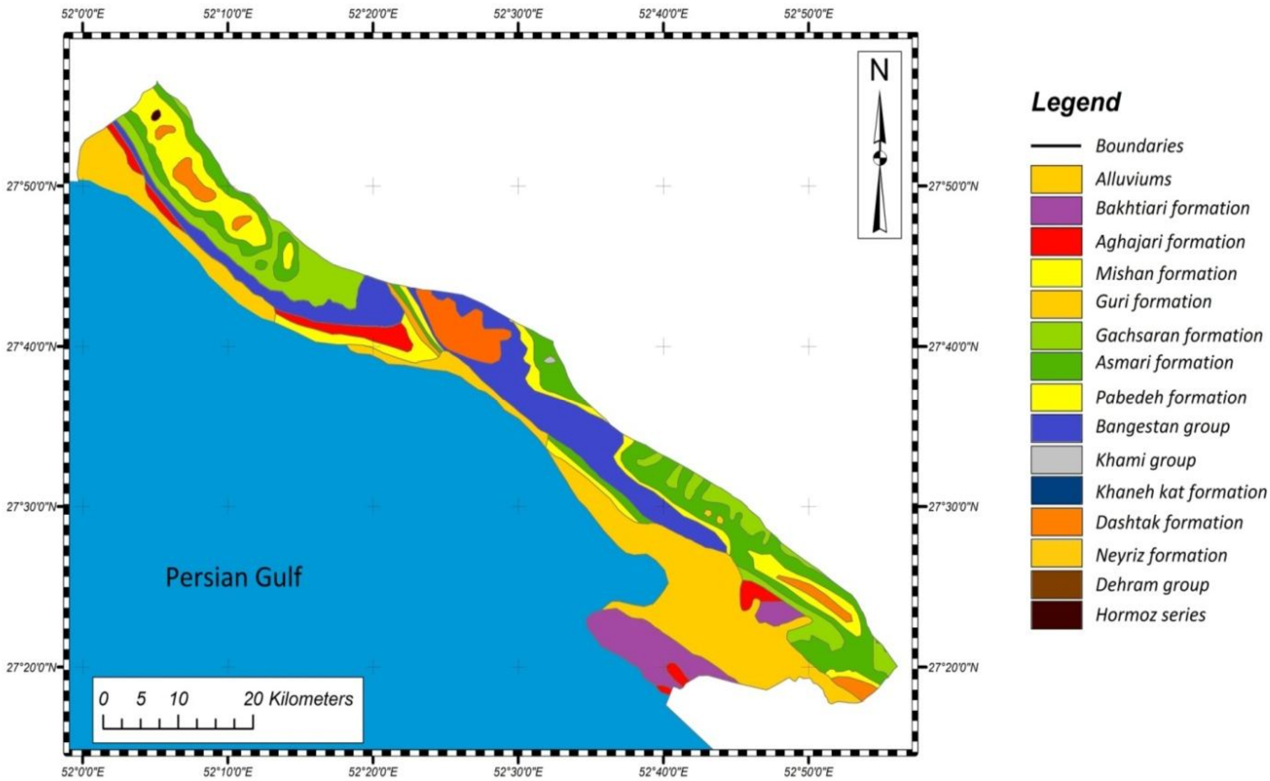

Figure. 3. The geological map of SPZ (adapted from GSI, 2009)

4. Slope stability assessment

Due to the presence of the SPZ in the Assalouyeh region, the construction materials are either extracted from the sea (Persian Gulf) or from the north of the region which is related to the southern side of the Assalouyeh anticline and to the Aghajari and Mishan formations (Azarafza et al., 2017c). This land development operation is utilised for roads, corridors, sites, etc., especially for SPZ which leads to failures or local instability in these areas.

Application of the Q-slope classification system for the analysis of these slopes with acceptable accuracy and simplicity is an appropriate way to estimate the slope instabilities. For this purpose, 55 cases of the sensitive and important rock slopes in the SPZ have been selected and sustainability analyses have been conducted to specify the empirical stability state of these slopes as a primitive assessment. During field survey in the SPZ, rock mass geometrical features and discontinuity properties were recorded for each slope and the Q-value indices were provided as indicated in Tables 2 to 7. Also, for the evaluation of the geological engineering characteristics from these slopes, rock sampling was conducted. By using field investigation data and geotechnical tests on samples, the stability of the slopes were assessed. The results of the stability analyses were plotted on a Q-slope stability chart (Barton and Bar, 2015; 2017) as illustrated in Fig. 5 and the slope datasets based on Q-slope are presented in Figs. 6 and 7. According to the results of the study conducted on the main slopes in SPZ and implementation of the Q-slope system, the major part of the slopes in terms of sustainability is located in critical/uncertain condition (25 cases) whereas 10 slopes are identified as unstable and 20 slopes are classified as stable.

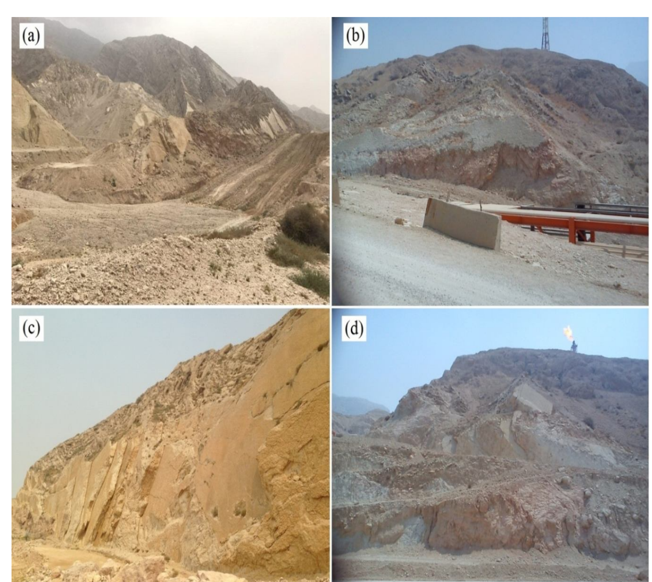

Figure. 4. A view of some studied slopes

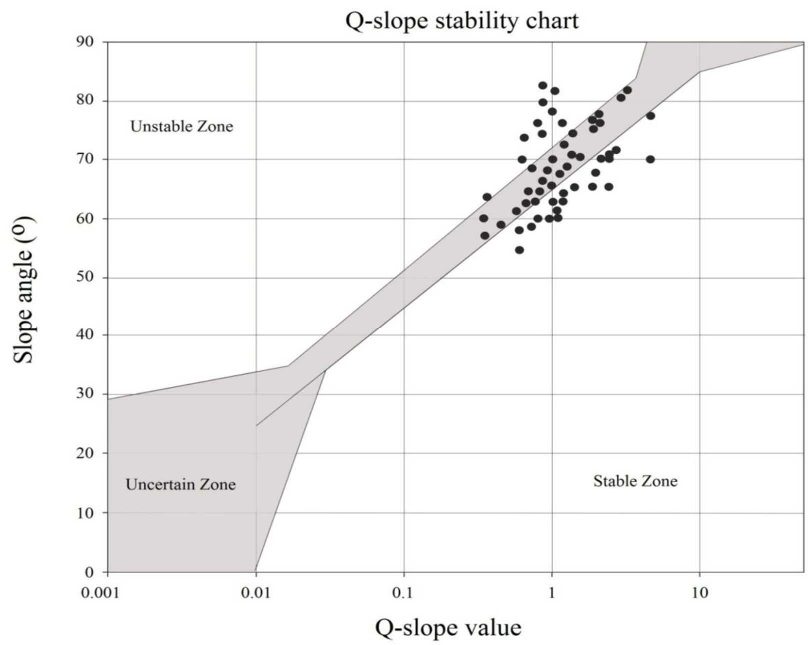

Figure. 5. Results of the Q-slope stability assessment on the stability chart

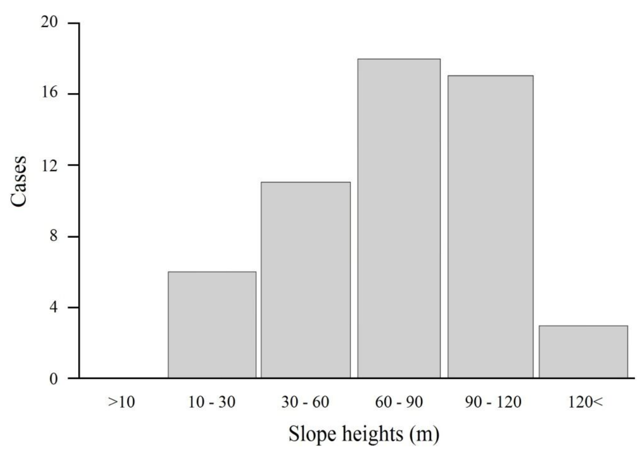

Figure. 6. Elevation changes of studies slopes

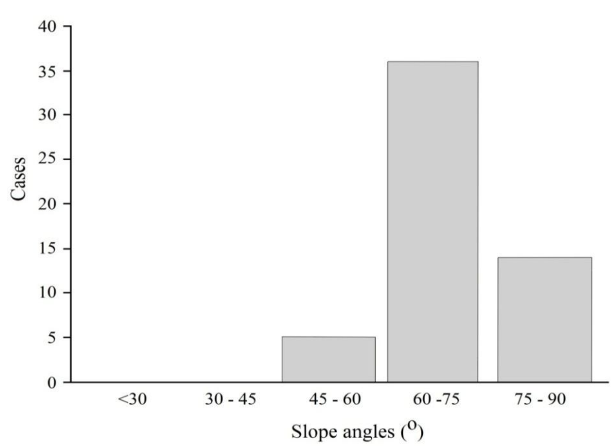

Figure. 7. Angle changes of studies slopes

5. Conclusions

Rock slope stability based on Q-slope (and all empirical methods) is used for engineering judgments and to solve problems which are directly related to the experiences of the geo-engineers. Although involving the engineering judgments on the assessment makes high sensitivity and tolerance on designs, there is a very fast way to achieve important results and acceptable classifications in engineering operations. On the other hand, using precise methods and interpretive analyses in the early stages is very time consuming without any flexibility in design. Thus, application of empirical methods and classification systems during the executive and preparatory stages can prove to be effective.

In this study the newest empirical classification system was utilised for stability analyses and for the description of the discontinuous rock-slope conditions located in the South Pars Zone in the south of Iran. According to the landslide susceptibility assessment of the SPZ, this area is located in the most risk-able region that is subjected to various types of slope failures which need urgent attention for stabilisation. For this purpose, 55 susceptible slopes in SPZ were identified and their stability analysis was conducted by the Q-slope system. According to the results of the assessment, the major part of these slopes is in critical/uncertain condition (25 cases) whereas 10 slopes are unstable and 20 slopes are stable.

Acknowledgements

The authors wish to thank the South Pars Gas Complex management for giving permission to perform field studies.

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