Q-Slope System for Assessing the Stability of Rock Slopes in Selected Area, Mergasur Town, Erbil, Kurdistan Region, NE Iraq

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Abstract

Rock slope, slope height, rock discontinuity orientations, and undesigned excavated slopes are the primary contributing factors to the instability of the road in the mountain area. The eight rock slopes in Mergasur town, NE-Iraq, were chosen to be assessed for stability using the kinematic approach with DIPS v6.008 software to determine the type of slope failure and the Q-slope system applied to determine the stability condition. This is an efficient approach for classifying rock slope engineering. According to the kinematic results, stations 1–5 and 7 may have planar sliding, whereas stations 4–8 may experience wedge sliding, stations 1, 2, and 7–8 may experience flexural toppling, and station 3 may have direct toppling. In accordance with the Q-slope system results, stability conditions are determined by projecting the Q-slope and slope angle values on the Q-slope chart.

Keywords: Rock slope stability; Q-slope system; Kinematic approach; Mergasur Town, Iraq

1. Introduction

One of the most frequent natural disasters is slope failure, which can cause serious property damage as well as fatalities. Because of alterations to the characteristics of the rock mass and environmental factors like hydrological effects and seismic activity, assessing the rock slopes in most road cuts especially in rough terrain runs the risk of experiencing instability issues (Pantelidis, 2009).

There are occasionally many rock collapses along this road, which puts people at risk and causes traffic congestion (especially during the wet seasons). Regrettably, there are just private records available for this; there is no public property. The road cut and slope instability problem are significantly impacted by the slope height, slope angle, and discontinuous orientations. Additionally, the road’s curvature may have an impact on the stability risk of rock slopes, particularly in rugged mountainous regions (Hoek and Bray, 1981).

The rock slope stability along major roads in cities and mountain terrain, has been examined by many researchers including Qader and Syan (2021), Siddique et al. (2020), Azarafza et al. (2017), and Jordá-Bordehore (2017). Furthermore, numerous studies regarding engineering geology and rock slope stability have been applied in Iraq, including the kinematic technique by Mamlesi (2010), the landslide possibility index (LPI) method by Qader et al. (2020), Hamasur (2013), Hostani and Hamasur (2022), and the Q-slope system by Hamasur (2022). According to Azarafza et al. (2017), a wide range of engineering systems have been created for slope engineering design and general assessments. Engineering systems were designed to determine the essential parameters and their rating value using empirical formulae in order to determine the rock mass final value. The aim of this study is to evaluate

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the stability of a selected rock slopes by using Kinematic analysis to estimate the potential stability of rock slopes, and the Q-slope system (Bar and Barton, 2017) will be implemented for determining the stability condition of rock slopes.

The study location is 144 km northeast of Erbil, in northern Iraq. The site is near Lerabire village in the Barzan sub-district, between latitudes 36°54′52.18″ and 36°56′7.92″N, and longitudes 44°11′2.46″ and 44°09′19.74″ E, as in Figs. 1 and 2.

Fig. 1. The study area’s rock slope stations are indicated on a topographic map

2. Geological Setting

Two formations that range from the Early Cretaceous to the Middle Cretaceous are the exposed geological units in and around the research area. According to Figs. 8 to 11 (rock slope stations no. 5 to 8), the oldest unit of the lower Cretaceous age and the Qamchuqa Formations are classified as the core of the majority of the anticlines in the research area. Bekhme Formation is characterized as an Upper Cretaceous age with carbonate and impure carbonate rocks as Figs. 4–8 respectively (rock slope stations no. 1–4) (Qader et al., 2020). These geological units from older to younger are shown in Fig. 2.

According to Al-Sinawi and Al-Qasrani (2003), the study area is located in the imbricated zone. Shirin and Bradost anticlines are distinguished by the presence of numerous large faults of various forms dispersed throughout the anticline (Jassim and Buday, 2006; Jassim and Goff, 2006; Fouad, 2010). Omar (2005) explained that different fault types seem particularly in the south western limb. Both anticlines are an asymmetrical, cylindrical anticlinal fold with trends NW-SE. The northwest plunge of Bradost anticline is in en-echelon position with Shirin anticline in southeast Barzan town near Dore village (Qader et al., 2020). While the southeast plunge is located in Rawanduz town, it interferes with the Korek anticline and is an en-echelon with the Handrin anticline (Balaki and Omar, 2019). The Gali
Rukuchk is formed from the cutting (dissection) of the Shirin and Bradost anticlines (Ahmed, 2019). The majority of the research area is situated at the Shirin Anticline's south-eastern plunge.

**Fig. 2.** a) Location map of the research area with a map of Iraq's tectonic divides (after Fouad, 2015), b) Geological and structural map of the investigated area (after Zebari et al., 2019)

### 3. Materials and Methods

In tunneling and underground mining applications, the Q-system (Barton et al., 1974) and rock mass rating (RMR) (Bieniawski, 1976; Bieniawski, 1989) are used to provide suitable reinforcement for the excavation slope. The SMR (slope mass rating) (Romana, 1985; Sullivan, 2013) was established to estimate the support for excavated slopes, according to Bar and Barton (2017). Furthermore, the long-term stable slope angle is not indicated in the above rock engineering systems.

The engineering geological survey was carried out in July and August of 2021 for eight rock stations; the four stations (1, 2, 3 and 4) are within the Bekhma Formation and the other stations (5, 6, 7 and 8) are situated in Qamchuqa Formation. The field measurements are based on dip direction and dip angle manners for all discontinuities attitude and slopes. Slope and discontinuities attitude, spacing, and condition of discontinuities and groundwater are measured and described in the field. The strength of the rock sample was determined using a point load test by Broch and Franklin (1972) as recommended by ISRM (1985). The basic purpose of the study is to evaluate the stability of rock slopes using kinematic analysis and the Q-slope system.

The kinematic approach is a simple practice for identifying potential types of failure using stereographic analysis (Raghuvanshi, 2019; Qader and Syan, 2021). Because of the angular correlations between discontinuities and the slope surface, they are also to be used to determine the orientation of a jointed rock mass (Hoek and Bray, 1981; Yoon et al., 2002). When a wedge block slides within the line where two planes connect, it can cause a wedge failure; Markland (1972) is one of the kinematic analytic techniques for evaluating this risk. Hocking (1976) modified and explained the Markland method.
DIPS v6.008 software was used to graphically evaluate the field data (Rocscience, 2015). In the field, friction angles of 31°-33° were estimated using the Bruce et al. (1989) tilting approach.

The Q-slope system is built on Barton's Q-system, which was developed for underground stability analysis and support system design (Barton et al., 1974; Barton and Grimstad, 2014). This method allowed geologists to measure rock slope stability and make slope angle alterations in situ for rock mass. It was used to reduce slope support or bench-width supplies. The Q-slope approach will be applied to all types of rock slope failures (Barton and Bar, 2015). Geometrical features, strength conditions, and the in-situ stress field, according to Nikoobakht and Azarafza (2016), are significant benefits of this classification that can be applied to open-pit mining, slope geometrical stabilizations and slope cuttings.

The Q-slope system has a six parameters as the standard Q-system (RQD, Jn, Jr, Ja, Jw, and SRF), but for slope assessment they are modified as RQD, Jn, Jr, Ja, Jwice, and SRFslope (Barton and Bar, 2015; Bar and Barton, 2016 & 2017). The Q-slope formula for assessing slope stability is as follows:

\[
Q \text{-Slope} = \frac{(RQD/Jn)}{(Jr/Ja)}\times (Jwice/SRFslope)
\]  

Regarding the equation, Jwice indicates the number of environmental and geological conditions, SRF slope represents the strength reduction factors, SRFa refers to the number of physical conditions, SRFb refers to the number of stresses and strengths, and SRFc refers to the number of major discontinuities. The other Q-system parameters stay constant. In addition, the orientation factor is the O-factor, which includes the Jr/Ja ratio. The three primary parts of the Q-slope and Q-system are defined as block size (RQD/Jn), inter-block shear strength (Jr/Ja), and active stress or external factors (Jwice/SRFslope). RQD stands for Rock Quality Designation. Jn: Joint set number. Jr: Joint roughness number. Ja: Joint alteration number. O: Orientation factor.

For slopes less than 30 m in heights, Barton and Bar (2015) developed an equation for the steepest angle (β), which eliminates the need of reinforcements and supports. According to Bar and Barton (2017), equation 2 is suitable for all types of slope heights, and it also matches slopes ranging from 35° and 85° degrees:

\[
\beta = 20 \log_{10} Q – \text{slope} + 65°
\]  

4. Results and Discussion

To determine the discontinuity pattern, eight rock slopes were chosen in the study area based on structural geological appearance, topographic conditions, and lithology characteristics. Because of their geotechnical properties, the rock slopes in the research area have been thoroughly investigated. The rock slopes in the Bekhma and Qamchuqa formations are made of a massive bed of limestone and a dolomitic limestone bed.

As shown in Table 1, the rock slopes have a variable slope angle with a developed system of discontinuities. A Silva compass was used in the field to measure the slope and discontinuity attitude. The friction angles of discontinuity failure surfaces have been calculated which are 31° and 32° for limestone and 33° for dolomitic limestone. Table 2 contains the findings of all kinematic analyses.
Table 1. Field data and measurements in the rock slopes

<table>
<thead>
<tr>
<th>Station No.</th>
<th>Rock type</th>
<th>Formation</th>
<th>Dip Direction / Dip amount</th>
<th>Friction angle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Slope</td>
<td>Bedding</td>
</tr>
<tr>
<td>1</td>
<td>Limestone</td>
<td>Bekhma</td>
<td>052º/69º</td>
<td>051º/35º</td>
</tr>
<tr>
<td>2</td>
<td>Limestone</td>
<td></td>
<td>053º/62º</td>
<td>060º/35º</td>
</tr>
<tr>
<td>3</td>
<td>Limestone</td>
<td></td>
<td>050º/38º</td>
<td>035º/37º</td>
</tr>
<tr>
<td>4</td>
<td>Limestone</td>
<td></td>
<td>242º/44º</td>
<td>360º/09º</td>
</tr>
<tr>
<td>5</td>
<td>Limestone</td>
<td></td>
<td>285º/53º</td>
<td>030º/65º</td>
</tr>
<tr>
<td>6</td>
<td>Dolomitic</td>
<td>Qamchuqa</td>
<td>278º/80º</td>
<td>060º/12º</td>
</tr>
<tr>
<td>7</td>
<td>Dolomitic</td>
<td>Qamchuqa</td>
<td>240º/70º</td>
<td>060º/68º</td>
</tr>
<tr>
<td>8</td>
<td>Limestone</td>
<td></td>
<td>292º/73º</td>
<td>091º/61º</td>
</tr>
</tbody>
</table>

For conducting kinematic analysis on the examined rock slopes, the Markland test, average attitude of slopes and discontinuities, and internal friction angle of rock discontinuities were utilized. The DIPS v6.008 software was used to identify probable failure zones; additionally, all rock slopes, discontinuities (bedding planes and joints), and potential failure zones were represented in pink color in stereographic projection as poles. The following kinematic study of rock slopes was determined:

- Potential for planar sliding at stations no. 1 to 5 and 7 as shown in Figs. 3a, 4a, 5a, 6a, 7a and 9a respectively, also 3b, 4b, 5b, 6b, 7b and 9b respectively.
- Potential for wedge sliding at stations no. 4 to 7 and 8 as shown in Figs. 6a, 7a, 8a, 9a and 10a respectively, also 6c, 7c, 8c, 9c and 10b respectively.
- Potential for flexural toppling at stations no. 1, 2, 7 and 8 as shown in Figs. 3a, 4a, 9a and 10a respectively, also 3c, 4c, 9d and 10c respectively.
- Potential for direct toppling at station no. 3 as shown in Fig. 5a and 5c.

In kinematic analysis stereographic projection figures and tables have the following: slope face = SF; bedding plane = So; joint set no.1 = J1; joint set no.2 = J2; joint set no.3 = J3; intersection between two discontinuity sets = I. The failure mode (planar sliding; PS, wedge sliding; WS, flexural toppling; FT and direct toppling; DT) direction is northeast to southwest ranging from 28º to 320º, represented on the stereonet as an arrow also in Table 2.
Fig. 3. a) Field image with distinct discontinuity sets for station no. 1. b) PS is shown on the bedding plane (So) in the kinematic analysis for station no. 1. c) FT about J2 is visible in the kinematic analysis for station No. 1
Fig. 4. a) Field image with distinct discontinuity sets for station no. 2. b) PS on the (So) is seen in kinematic analysis for station no. 2. c) shows FT about J2
Fig. 5. a) Field image with distinct discontinuity sets for station no 3. b) PS on the (So) is seen in kinematic analysis for station no. 3. c) DT seen in intersected release planes J1 and J2 (I (J1 & J2)) in kinematic analysis for station no. 3 with the aid of so as a basal plane.
Fig. 6. a) Field image with distinct discontinuity sets for station no 4. b) PS on the (J1) is seen in kinematic analysis for station no 4. c) WS on I (J1 and J2) is seen in kinematic analysis for station no 4.
Fig. 7. a) Field image with distinct discontinuity sets for station no 5. b) PS on the (J2) is seen in kinematic analysis for station no 5. c) WS on (So & J2) is seen in kinematic analysis for station no 5.

Fig. 8. a) Field image with distinct discontinuity sets for station no 6. b) WS on J1 and J2 is seen in kinematic analysis for station no 6.
Fig. 9. A) Field image with distinct discontinuity sets for station no. 7. B) PS on the J1 is seen in kinematic analysis for station no. 7. C) WS on (J1 & J2) is seen in station no. 7. D) FT about So is seen in kinematic analysis for station no. 7.
Fig. 10. a) Field image with distinct discontinuity sets for station no 8. b) WS on (J1 & J2) is seen in kinematic analysis for station no. 8. c) FT about So is seen in kinematic analysis for station no. 8

Table 2. Failure types and their directions in rock slope kinematic analyses

<table>
<thead>
<tr>
<th>Station no.</th>
<th>Formation</th>
<th>PS direction</th>
<th>WS direction</th>
<th>FT direction</th>
<th>DT direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Bekhma</td>
<td>√ (51º)</td>
<td>-</td>
<td>√ (61º)</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>√ (60º)</td>
<td>-</td>
<td>√ (68º)</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>√ (035º)</td>
<td>-</td>
<td>-</td>
<td>√ (28º)</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>√ (229º)</td>
<td>√ (241º)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>√ (266º)</td>
<td>√ (320º)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>-</td>
<td>√ (255º)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>Qamchuqa</td>
<td>√ (250º)</td>
<td>√ (195º)</td>
<td>√ (240º)</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>-</td>
<td>√ (264º)</td>
<td>√ (271º)</td>
<td>-</td>
</tr>
</tbody>
</table>

The six parameters described above determined Q-slope values for all rock slopes. Based on equation no. 3 the RQD determined for all rock slopes as shown in Table 3.

\[ \text{RQD} = 110 - 2.5 \times Jv \quad (3) \]
UCS was calculated indirectly using point load tests performed in accordance with ISRM (1985) using a strength conversion factor of 22.5 (k=22.5), which is applicable for Limestone and Dolomitic limestone rock types (Bieniawski, 1975). Table 4 shows the rock strength results UCS rate with the maximum principal stress (σ1) is required for determining the SRFb-stress. The climate condition is a semi-arid condition in the study area, based on the fact that most landslides occur during rainy seasons, and according to that the environmental condition (Jwice) will be a wet condition. Table 5 shows the rock properties for Q-slope criteria on the studied rock slopes. Furthermore, the rock mass characteristics for each rock slope were identified through a comparison of Q-slope parameter characteristics (Table 5) with the Q-slope tables of Barton and Bar (2015).

Table 3. The volumetric joint count (Jv) and RQD in the rock slope stations

<table>
<thead>
<tr>
<th>Station No.</th>
<th>Formation</th>
<th>Jv (joints/m3)</th>
<th>RQD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Bekhma</td>
<td>4.93</td>
<td>98</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>3.95</td>
<td>100</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>6.99</td>
<td>93</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>4.07</td>
<td>100</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>4.55</td>
<td>99</td>
</tr>
<tr>
<td>6</td>
<td>Qamchuqa</td>
<td>4.25</td>
<td>99</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>4.28</td>
<td>99</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>4.06</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 4. Point load test results and the UCS of rocks at stations 1 to 8

<table>
<thead>
<tr>
<th>Station No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Formation</td>
<td>Bekhma</td>
<td></td>
<td></td>
<td></td>
<td>Qamchuqa</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D (mm)</td>
<td>41</td>
<td>39</td>
<td>41</td>
<td>42</td>
<td>43</td>
<td>39</td>
<td>43</td>
<td>43</td>
</tr>
<tr>
<td>W (mm)</td>
<td>49</td>
<td>52</td>
<td>62</td>
<td>53</td>
<td>53</td>
<td>51</td>
<td>51</td>
<td>58</td>
</tr>
<tr>
<td>F (KN)</td>
<td>15.1</td>
<td>17</td>
<td>15.1</td>
<td>13</td>
<td>15</td>
<td>14.2</td>
<td>11</td>
<td>16.6</td>
</tr>
<tr>
<td>F (MN)</td>
<td>0.015</td>
<td>0.017</td>
<td>0.0151</td>
<td>0.013</td>
<td>0.015</td>
<td>0.014</td>
<td>0.011</td>
<td>0.016</td>
</tr>
<tr>
<td>Area mm²</td>
<td>2009</td>
<td>2028</td>
<td>2542</td>
<td>2226</td>
<td>2279</td>
<td>1989</td>
<td>2193</td>
<td>2494</td>
</tr>
<tr>
<td>de²=(4DW/π)m²</td>
<td>0.003</td>
<td>0.00258</td>
<td>0.00324</td>
<td>0.00284</td>
<td>0.00290</td>
<td>0.00253</td>
<td>0.00279</td>
<td>0.00318</td>
</tr>
<tr>
<td>Is=F/de²</td>
<td>5.901</td>
<td>6.58038</td>
<td>4.66306</td>
<td>4.58446</td>
<td>5.16674</td>
<td>5.60432</td>
<td>3.93753</td>
<td>5.22494</td>
</tr>
<tr>
<td>F=(D/50)0.45</td>
<td>0.914</td>
<td>0.89422</td>
<td>0.91457</td>
<td>0.92454</td>
<td>0.93438</td>
<td>0.89422</td>
<td>0.93438</td>
<td>0.93438</td>
</tr>
<tr>
<td>UCS=(22.5*Is(50)) (Mpa)</td>
<td>121.4</td>
<td>132.396</td>
<td>95.955</td>
<td>95.367</td>
<td>108.623</td>
<td>112.758</td>
<td>82.781</td>
<td>109.847</td>
</tr>
<tr>
<td>UCS (Mpa)</td>
<td>≈ 121</td>
<td>≈ 132</td>
<td>≈ 96</td>
<td>≈ 95</td>
<td>≈ 109</td>
<td>≈ 113</td>
<td>≈ 83</td>
<td>≈ 110</td>
</tr>
</tbody>
</table>

Where: D=Diameter (distance between the two loaded points), W=Width of the rock block sample, A=W*D (Area of idealized failure plane), F=Force at failure, Is=Point load strength index, f=(size correction factor), UCS=uniaxial compressive strength.

For each rock slope in the study area, the Q-slope value and slope dip angle have been plotted on the Q-slope chart to establish the stability condition, as illustrated in Fig. 11. In addition, the rock slope stations no. 3, 4 and 5 have a stable slope, while the rock slope stations no. 1, 6, 7 and 8 have an unstable
slope, but station no. 2 have an uncertain slope. Furthermore, the steepness slope angle ($\beta$) not required supports and reinforcement was calculated using equation (2), and Table 6 presents the outcomes.

### Table 5. Description of the rock mass and main Q-slope characteristics in the study area

<table>
<thead>
<tr>
<th>Site no.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slope height</td>
<td>20</td>
<td>13</td>
<td>18</td>
<td>12</td>
<td>25</td>
<td>30</td>
<td>35</td>
<td>9</td>
</tr>
<tr>
<td>61 (MPa)</td>
<td>0.5</td>
<td>0.325</td>
<td>0.45</td>
<td>0.3</td>
<td>0.625</td>
<td>0.75</td>
<td>0.875</td>
<td>0.255</td>
</tr>
<tr>
<td>UCS ($\gamma_{\text{rock}} = 0.0245$ MN/m$^3$)</td>
<td>121</td>
<td>132</td>
<td>96</td>
<td>95</td>
<td>109</td>
<td>113</td>
<td>83</td>
<td>110</td>
</tr>
<tr>
<td>6c /61</td>
<td>242</td>
<td>406</td>
<td>213</td>
<td>317</td>
<td>174</td>
<td>151</td>
<td>95</td>
<td>489</td>
</tr>
<tr>
<td>Failure mode</td>
<td>PS, FT</td>
<td>PS, FT</td>
<td>PS, DT</td>
<td>PS, WS</td>
<td>PS, WS</td>
<td>WS</td>
<td>PS, WS, FT</td>
<td>WS, FT</td>
</tr>
<tr>
<td>RQD</td>
<td>98</td>
<td>100</td>
<td>93</td>
<td>100</td>
<td>99</td>
<td>99</td>
<td>99</td>
<td>100</td>
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<tr>
<td>Jn</td>
<td>F</td>
<td>F</td>
<td>F</td>
<td>F</td>
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<td>B</td>
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<td>C</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>C</td>
</tr>
</tbody>
</table>

Where: 61 = Maximum principle stress ( $\gamma_{\text{rock}} = 0.0245$ MN/m$^3$), UCS ($6c$) = Uniaxial compressive strength, PS = Planar sliding, FT = Flexural toppling, WS = Wedge sliding, DT = Direct toppling, (For Jn: F= Three joint sets) (For Jr: B= Rough or irregular, undulating, E= Rough or irregular, planar), (For Ja: C= Slightly altered joint walls, Non-softening mineral coatings, sandy particles, clay- free disintegrated rock, etc.), (For Jwince: Unsta.Com = Unstable structure competent rock, WE+IW = wet environment+ice wedging), (For SRFa: B= Loose blocks, signs of tension cracks and joint shearing, susceptibility to weathering), (For SRFb: F= Moderate stress-strength range ($6c / 61$: 50-200 or greater)), (For SRFc: L.VUnfa = L=Major discontinuity with little or no clay - Very unfavourable, VUnfa= Very unfavourable, Unfa= Unfavourable, c.= causing failure if unsupported)
**Table 6. Field data and result of the Q-slope system parameters**

<table>
<thead>
<tr>
<th>Study site No.</th>
<th>RQD</th>
<th>Jn</th>
<th>Jr</th>
<th>Ja</th>
<th>O-factor</th>
<th>Q-slope value</th>
<th>SRFslope</th>
<th>Q-Slope value</th>
<th>Slope angle</th>
<th>Stability condition of slope</th>
<th>Stable slope angle without support</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>98</td>
<td>9</td>
<td>1.5</td>
<td>2</td>
<td>0.4</td>
<td>0.5</td>
<td>5</td>
<td>1</td>
<td>4.5</td>
<td>PS: 0.326 FT: 0.490</td>
<td>Unstable slope</td>
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<tr>
<td>2</td>
<td>100</td>
<td>9</td>
<td>1.5</td>
<td>2</td>
<td>0.4</td>
<td>0.5</td>
<td>5</td>
<td>1</td>
<td>4.5</td>
<td>PS: 0.333 FT: 0.500</td>
<td>Slope stability uncertain</td>
</tr>
<tr>
<td>3</td>
<td>93</td>
<td>9</td>
<td>3</td>
<td>2</td>
<td>0.4</td>
<td>0.75</td>
<td>5</td>
<td>1</td>
<td>2.5</td>
<td>PS: 0.930 DT: 0.930</td>
<td>Stable slope</td>
</tr>
<tr>
<td>4</td>
<td>100</td>
<td>9</td>
<td>3</td>
<td>2</td>
<td>0.4</td>
<td>0.5</td>
<td>5</td>
<td>1</td>
<td>2.5</td>
<td>PS: 0.990 WS: 0.890</td>
<td>Stable slope</td>
</tr>
<tr>
<td>5</td>
<td>99</td>
<td>9</td>
<td>3</td>
<td>2</td>
<td>0.4</td>
<td>0.5</td>
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<td>5</td>
<td>2</td>
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</tr>
<tr>
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<td>1.5</td>
<td>2</td>
<td>0.4</td>
<td>0.5</td>
<td>1.5</td>
<td>4</td>
<td>5</td>
<td>PS: 0.206 WS: 0.550</td>
<td>Unstable slope</td>
</tr>
<tr>
<td>7</td>
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<td>3</td>
<td>2</td>
<td>0.4</td>
<td>0.5</td>
<td>2.5</td>
<td>8</td>
<td>8</td>
<td>PS: 0.14</td>
<td>Unstable slope</td>
</tr>
<tr>
<td>8</td>
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<td>9</td>
<td>1.5</td>
<td>2</td>
<td>0.4</td>
<td>0.5</td>
<td>5</td>
<td>1</td>
<td>8.5</td>
<td>PS: 0.208</td>
<td></td>
</tr>
</tbody>
</table>

Where: PS=Planar sliding, FT=Flexural toppling, WS=Wedge sliding, DT=Direct toppling, OT=Oblique toppling.
5. Conclusions

The best practical method for evaluating the rock slope stability, which includes joint sets is the kinematic approach. In addition, planar sliding in the rock slopes of stations no. 1, 2, 3, 4, 5, and 7, wedge sliding in the rock slopes of stations no. 4, 5, 6, 7, and 8, flexural toppling in the rock slopes of stations no. 1, 2, 7 and 8, and direct toppling in the rock slope of station no. 3. The major types of failure in rock slope stations are planar and wedge sliding. The Q-slope system result concluded that the rock slope stations no. 3, 4 and 5 are in stable slope condition, whereas the rock slope stations no. 1, 6, 7, and 8 are in unstable slope condition; nevertheless station no. 2 is uncertain slope stability.

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References


