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Slope Stability and Hazard Assessment of Rock Cut Slopes along Al-Husiah Road, Shara'ab Al-Rawnah District, Northwest Taiz, Yemen

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ABSTRACT

Yemen features rugged terrain with table lands and a narrow coastal plain. The road networks pass across a variety of hill slopes, ridges, and mountain ranges and are prone to slope failure and landslides on both sides of the road, especially during rainy seasons.Topography, geological formations, variable lithostratigraphy, and tectonics, in addition to human activities, all contribute to the terrain's susceptibility to groundmass wasting and movement along road cut slopes. Among thousands of road networks in Yemen, Al-Husiah road located in Shara'ab Al-Rawnah region, North Taiz governorate, southwest Yemen is witness to groundmass movements along structural discontinuities and the different types of failures such as toppling, wedge and planar failures, and fall of rock fragments and blocks.The present research work is aimed at evaluation of the stability of 12 rock cut slopes along the Al-Husiah road (about 5.76 km long) using the original Slope Mass Rating (SMR) system and Landslide Possibility Index (LPI) in addition to kinematic slope stability analysis.The obtained results from applying SMR system at 12 rock cut slope locations indicate that the stability of the slope can be classified as "Stable" to "Completely Unstable" (SMR class II to V), and LPI values indicate that the investigated rock cut slopes fall in the class of "High" (H) Hazard zones. Kinematic analysis performed to predict the site and type of possible failures facilitated the identification of potentially vulnerable slopes. The remedial measures to minimize the risk of slope failures in 12 slope locations have been suggested based on calculated average SMR values and field observations.

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1. INTRODUCTION

In Yemen, approximately two thirds of the population reside in rural areas in which most of the natural landscape is characterized by gentle to steep hill slopes, ridges and mountainous terrain. The road network laid in the rugged terrain is considered to be a critical component of a community's infrastructure, especially in rural areas and plays an important role for transportation, public conveyance and other socio-economic activities. However, the road network developed in the mountainous and hilly terrain is usually vulnerable to slope failure as the rocks exposed are highly weathered and thus landslide along both the sides of the roads are common, especially during the rainy seasons. The stability of rock slopes and failure problems are mainly governed by the local geological and reengineering characteristics of the slope forming mass, structural discontinuity characteristics, weathering conditions, strength of slope materials, slope geometries and infiltrated rainfall as well as by human activities.

Rock cut slopes along Al-Husiah road are affected by groundmass movements along structural discontinuities (joints and bedding planes) and the different types of failures were recorded such as toppling, wedge and planar failures, and fall of rock fragment. The roadside cut slope failures pose risks to the safety of the traveling public, transportation, infrastructure, and to the environment, especially during heavy rainfall periods. For these reasons the sector of this road was selected for the detailed study.

Over the last several decades, various geomechanical classification systems such as rock mass rating (RMR; Bieniawski, 1976, 1979, and 1989), rock mass strength (RMS; Selby, 1980 and 1982; Moon and Selby, 1983; Moore et al. 2009), slope mass rating system (SMR; Romana, 1985 and Romana, et al. 2003), slope rock mass rating

(SRMR; Robertson, 1988; Singh et al. 2013), mining rock mass rating (MRMR; Laubscher 1990), modified mining rock mass rating (MRMR; Haines and Terbrugge, 1991), landslide possibility index (LPI; Bejerman, 1994; 1998), natural slope methodology (NSM; Shuk, 1994), Chinese slope mass rating (CSMR; Chen, 1995), modified rock mass rating (M-RMR; Ünal, 1996), rock slope deterioration assessment (RDA; Nicholson and Hencher, 1997; Nicholson et al. 2000; Nicholson, 2002, 2003 and 2004), slope stability probability classification (SSPC; Hack, 1998; Hack et al. 2003; Das et al. 2010; Canal and Akin 2016; Li and Xu 2016), modified slope stability probability classification (SSPC; Lindsay et al. 2001), volcanic rock face safety rating (VRFSR; Singh and Connolly, 2003), continuous rock mass rating (Sen and Sadagah, 2003; Tomás et al. 2007; Umrao et al. 2011; Sarkar et al. 2016), Slope stability rating system (SSR; Taheri and Tani, 2007 and 2010), the alternative rock mass classification system proposed by Pantelidis (2010), fuzzy slope mass rating (FSMR; Daf-taribesheli et al., 2011), new slope mass rating (NSMR; Singh et al., 2013), and Q-slope Method (Barton and Bar, 2015; Bar and Barton, 2017) have been proposed by many researchers to assess the behavior of a rock mass of natural and modified slopes These geomechanical classification systems are based on the assessment of rock mass in the field and determination of selected mechanical properties in the laboratory.

In the present work, the stability at 12 rock cut slopes was evaluated using the original slope mass rating (SMR) systemproposed by Romana (1985), whilethe degree of hazard was assessed by employing the Landslide Possibility Index (LPI) proposed by Bejerman (1994).The type of potential failure mechanism along discontinuity planes (structurally controlled failure) was identified based on Kinematic analysis method. The characterization of rock mass is also presented in this research work and the corresponding remedial measures for structurally-controlled rock slope failures are suggested based on the average SMR values and field observations.

SMR was determined by adding four adjustment factors to the basic rock mass rating (RMR-$_{89}$) system introduced by Bieniawski (1989). The adjustment factors rely on the discontinuity–slope relationship and the method of excavation (Romana, 1985). The degree of hazard was assessed by making use of the LPI proposed by Bejerman (1994). The LPI system is based on field quantification for ten key characteristics: Slope height, slope angle, grade of fracture of the rock mass, grade of weathering of the rock mass, gradient of the discontinuities, spacing of the discontinuities, orientation of the discontinuities, vegetation cover, water infiltration and inventory of landslides.

2. STUDY AREA

2.1 Location, Topography and Hydrology

The study area forms a part of the 5.76 km long Al-Husiah road located in the Shara'ab Al-Rawnah region, 29 kilometres north of Taiz governorate in southwest Yemen. The investigated rock cut slopes are bounded between Longitudes; 43° 46´ 52´´ and 43° 48´ 70´´ E., and Latitudes; 13° 45´ 54´´ and 13° 46´ 52´´ N. (Fig. 1).Topographically, the area is characterized by hilly- mountainous terrain with sharp summits, moderate to steep slopes and small valleys, at places with cliffs or escarpments. The elevations of the landforms vary from about 900 m to 1720 m amsl while the elevations of the examined road segment that passes through the topographic features range from 960m to 1450m amsl.These characteristics make the cut slopes along the road vulnerable for landslide hazards, posing a risk to the road and nearby residents' properties (Fig. 1).Hydrologically, the Shara'ab Al-Rawnah region is adjacent to the northwest of the upper Wadi Rasyan catchment which is characterized by the bimodal annual rainfall; the first season extends from April to June with peak in May and the second season is from August to October with peak in September. During the month July, there is less rainfall while the dry months begin in mid-October and end in mid-March. Rainfall record obtained from three rain gauge stations (Ussayfra/79-03, NWRA/ 98-04 and Taiz airport/76-79 & 83-89) located in and around Taiz city reveals that the average annual rainfall in the study area is about 520 mm (Al-Qadhi, 2007 & 2017). Streams in the region, like in any arid areas, are of ephemeral type and the principal source of water is rainfall runoff. During the rainfall periods, the meteoric water flows from high lands over steep slopes into natural flow channels which are in most cases are connected together forming the Wadis.

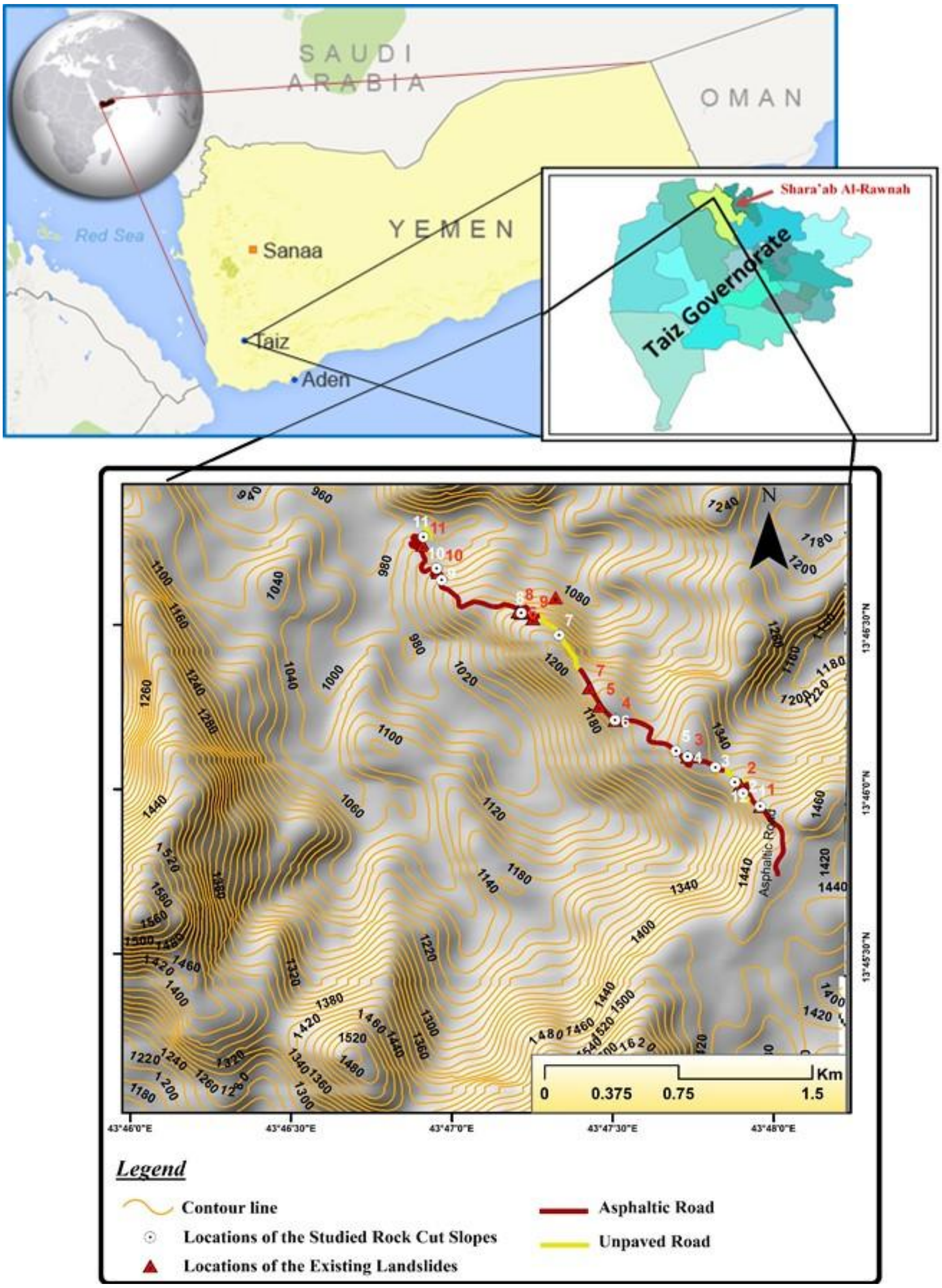

Figure 1: *3D Model (DEM) illustrates the location of studied road and the topographic features*

2.2. Geology of the area

The study area is covered by three main litho units (Fig.2). These are from bottom upwards:

1. Amran limestones

The Amran limestone belongs to Amran Group of sediments of Jurassic age and is exposed as extended beds from southeast to northwest direction (Fig.2). The formation consists of limestone intercalated with shales and randomly evaporates with clearly defined beddings (Malek, et al., 2021). The limestone is dark to light gray in color, fossiliferous and silicification phenomenon is observed. The silicification process of the limestone in study area is attributed to the process of metasomatism caused by volcanism during Tertiary times (Malek, et al., 2021). The limestone beds are locally affected by mesoscopic folding with fold axes trending in NE to SE direction.

2. Al-Tawillah sandstones.

Detailed field work carried out by the present workers reveal that the Tawillah Group is represented by sequence of two parts; (1) the middle and lower parts are characterized by fine to coarse-grained, moderately to poorly sorted and by the presence of conglomeratic sandstone with interbedded grey, green and red colored shales. The degree of roundness of the sand grains ranges from angular to subrounded, and (2) the upper part is characterized by fine- to medium grained, moderately to well sorted conglomerate layers with abundant kaolinite debris.

Al-Tawillah sandstone which is Upper Cretaceous in age is rested unconformably on the Amran limestone and is unconformably overlain by the Tertiary volcanic rocks (Fig.2). The unconformity is marked by a conglomerate bed in some locations, while in other areas, the contacts between Al-Tawillah sandstone and Amran limestone are sharp and irregular (Bagash, et al. 2018).

3. Tertiary volcanic rocks

The volcanic rocks in the area are represented by alternated succession of basalt flows and silicic rocks. These rocks are exposed in the southeast, west, southwest and northeastern parts of the study area (Fig.2). The basalt flows vary in their colors (from dark gray in fresh surface, to reddish brown in weathered exposures) and textures (from aphyric to porphyritic textures contain large plagioclase and olivine and phenocrysts). These flows are randomly to regularly fractured with fracture spacing up to 20 cm and sometimes exhibit colonnade and entablature structures.

The silicic rocks including rhyolite flows, ignimbrites and consolidated and unconsolidated ash deposits vary in their appearance (from sheet -bedding to domal hills), color (from light grey to light yellow) and grain size (from coarse grained porphyritic to very-fine grained) reflecting their composition variations. The basaltic dykes and sills showing discordant and concordant relation with Jurassic limestone and cretaceous sandstone, may represent the feeders for the basaltic flows. The dykes and sills are more resistant to weathering and erosion than the surrounding country rocks. Hence, at places, lithological ridges measuring several meters in height are formed. Generally, the rock units in the study area are faulted and sheared to varying degrees, and they have been subjected to different levels of weathering/ alteration and erosion.

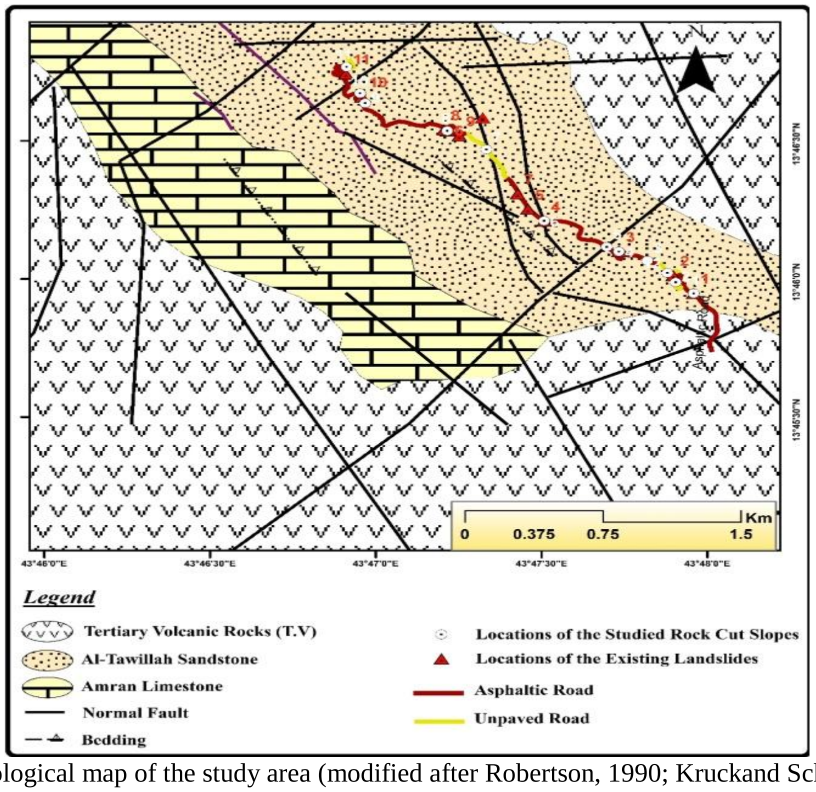

Figure 2: Geological map of the study area (modified after Robertson, 1990; Kruckand Schäffer, 1991)

The study region although is covered by Jurassic-Cretaceous formations and Tertiary volcanic rocks, the investigated road segment passes only through Cretaceous sandstone rocks of Al-Tawillah Group (Kt). The sandstone formations seen as elongated beds, strike on the rate trend of N40°W with dips ranging from 25° to 66° on the general direction N50°E (Malek, et al., 2021). Structurally, the Al-Tawillah sandstone rocks are traversed by several NNW-SSE oriented normal faults set which seems to be genetically and kinematically related to the Red Sea rifting system (Bagash, et al., 2018). The prime geologic and tectonic setting features of Al-Tawillah sandstone rocks encountered in the investigated road segment are listed below:

1. The sequence of rock formations along the road cut consists of varicolored, weathered, weakly welded shale/mudstone and conglomerate layers intercalated or alternating with the jointed sandstone beds (Fig. 3a). The sequence is trend in NW-SE direction with gently to moderately dipping to the northeast.
2. The sandstone beds also show variable color, mostly white to light yellow, medium to fine grained texture, and cross-bedding and graded bedding structures. The degree of grains sorting of sandstone varies from poor to well sorted.
3. The sandstone formation has well developed joints, faulted and sheared to varying degrees (Fig. 3b and c). As a consequence, the quality of the slope forming rock mass and the rock stability can be influenced by these geological structures.
4. The sequence of rocks has also been subjected to different levels of weathering/ alteration (moderate to high) resulting in the formation of residual soil (Fig. 3d).
5. The sandstone sequence is invaded by basic volcanic dykes and sills (Fig. 3e and 3f).
6. The existing heavy volcanic rocks directly above sandstone led to increasing the load stresses on the sandstone rocks (Fig.3g and see Fig. 2).

2.3 Landslides along the Al-Husiah road section

Al-Husiah road passes through the mountains, hills and valleys (see Fig. 1) composed of altered sequence of sandstones and mudstone/shale of Cretaceousage. This geological frame is usually very susceptible to cut slope failure along this terrain due to the prevalence of an alternate presence of weakly welded mudstone/shale rocks and of jointed hard sandstone rocks. Landslides have occurred along the analyzed road segment, causing damage to vehicular traffic as well as nearby properties such as dwellings and cultivated lands particularly during rainy periods. The locations of all recorded landslides are plotted on the geological map of the investigated area (see Fig.2), while field photographs of some landslides and rockfalls that have occurred along the Al-Husiah road near Albayda village, Al Qabina, Al Souhila, Al Mahjar, Madhal Shiep, Qareef Al medbaa and Dar Fangah are shown in Figure 4.

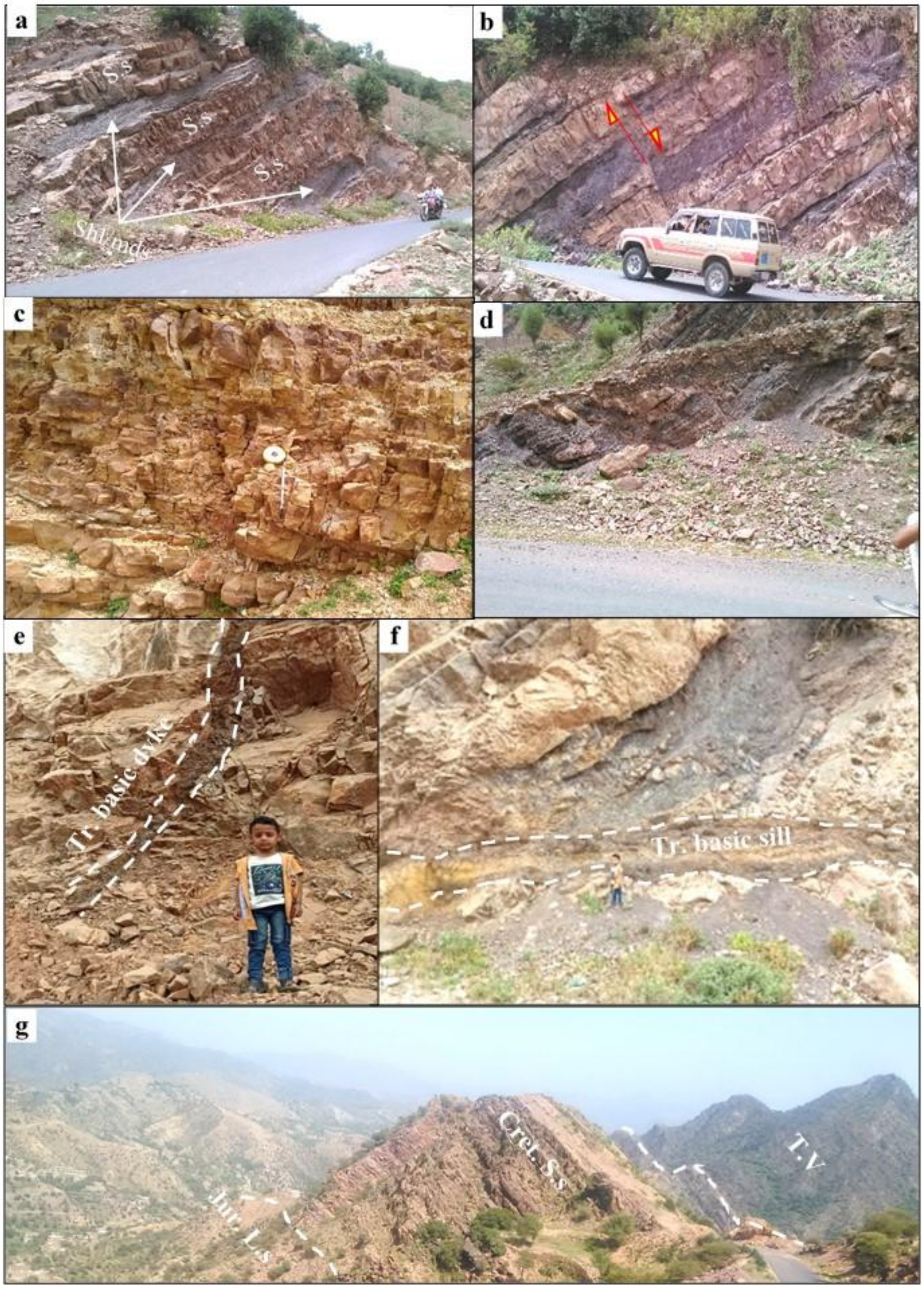

Figure 3: Field photographs showing(a) Cretaceous sandstone (S.s) beds alternating with mudstone/shale layers. Note the sandstone (S.s) sequence is gently dipping to the northeast and also the weathered, weakly welded of shale/mudstone layers, (b) normal fault in Cretaceous sandstone (S.s) rocks with a displacement of 1.20 m, (c) joints in Cretaceous sandstone (S.s), (d) weathering/ alteration of sandstone (S.s) and shale/mudstone (shl/mds) and formation of residual soil,(e) Cretaceous sandstone (S.s) sequence intruded by the Tertiary basic dyke and sill (f), (g) the sequence of sandstones is unconformably overlain by the Tertiary volcanic rocks (T.V) and underlain by Jurassic limestone (Jur. L.s).

Figure 4: Field photographs showing some landslides and rock falls in the sandstone rock cuts along the studied road; (1) sandstone rock fragments were slide forward of slope dip direction and opposite to the dip direction of bedding plane and accumulated as debris cone at the cut slope toe near Albayda village (landslide location no. 1; L.s. No. 1 and cut slope no.1),(2) the failure surface defined by a subvertical joint surface on which the small slide and rockfall occurred at Al Qabina area (L.s. No. 2 and cut slope no.2), (3) L.s. No. 3 (slope no.4) in a cutting of the road near Al Souhila (4) rock debris and blocks slide along bedding plane in Al Mahjar (L.s. No. 4 and slope no.5), (6) detached sandstone rock blocks at Madhal Shiep place (L.s. No. 6), (7) landslide no. 7 is located at Qareef Al medbaa, (8) the detached sandstone rock blocks were slide along the sandstone bedding plane in Dar Fangah location, also note that the ditched rock blocks caused obstruct the traffic for several hours (L.s. No. 8 and slope no.8).

3. METHODOLOGY

The site investigations were conducted at 12 different rock cut slope locations (Table 1 and see figures 1 and 2) along Al-Husiah road. At each location, the field scan line (tape) survey approach (Brady and Brown, 1985) was used to assess, characterize and measure all the parameters relevant to the basic rock mass rating (RMR $_{basic-89}$), rock slope kinematic analysis (RSKA), original slope mass rating (SMR) and evaluation of landslide possibility index (LPI).

Table 1: Locations of 12 rock cut slopes along Al-Husiah road

Slope location. No.	Location coordinates	
	Long. (deg.)	Lat. (deg.)
1	43° 47' 57.26"	13° 45' 57.94"
2	43° 47' 54.12"	13° 45' 59.87"
3	43° 47' 49.11"	13° 46' 04.94"
4	43° 47' 43.79"	13° 46' 06.47"
5	43° 47' 42.04"	13° 46' 07.97"
6	43° 47' 30.24"	13° 46' 13.08"
7	43° 47' 28.96"	13° 46' 13.90"
8	43° 47' 19.70"	13° 46' 28.53"
9	43° 47' 27.29"	13° 46' 15.64"
10	43° 47' 11.84"	13° 46' 33.37"
11	43° 46' 53.58"	13° 46' 43.51"
12	43° 46' 52.06"	13° 46' 44.49"

3.1 Determination of Basic Rock Mass Rating ($RMR_{basic-89}$)

Bieniawski's basic RMR (RMRbasic-89) was calculated (Bieniawski, 1989) using five basic input parameters (not adjusted for discontinuity orientation) that indicate rock mass conditions and discontinuity features. These parameters are: (1) Uniaxial Compressive Strength (UCS), (2) Rock Quality Designation (RQD), (3) Discontinuity Spacing (DS), (4) Discontinuity Conditions (DC), and (5) Groundwater Conditions (GWC). Each of these five classification parameters is given a rating value (R). $RMR_{basic-89}$ is expressed by the algebraic sum of the rating values of the five basic parameters as follows:

$RMR_{basic-89} = \sum_{i=1}^{5} Ri$(classification parameters)

$RMR_{basic-89} = R_{ucs} + R_{RQD} + R_{DS} + R_{DC} + R_{GWC}$ ……………………… ***(1)***.

where

- R_{ucs} is the rating value of uniaxial compressive strength (0–15),
- R_{RQD} is the rating value of rock quality designation, RQD (3−20),
- R_{DS} is the rating value of average joint space (5–20),
- R_{DC} is the rating value of joint wall conditions (0−30), and
- R_{GWC} is the rating value of groundwater conditions (0−30)

The uniaxial compressive strength (UCS) of rock material was estimated in the field based on the geological hammer and according to standard procedures [ISRM (1978c); CGS (1985); Marinos and Hoek (2001)]. RQD index was estimated from the volumetric joint count (Jv j/m^3). Volumetric joint count (Jv) is defined as sum of the number of joints per cubic meter (unit volume) and is measured from spacing's of the main joint/bedding sets within a volume of rock mass (Palmstrom, 1982, 1985, 1986; Sen and Eissa, 1991). Jv can be computed (Palmstrom, 2005) using the following Equation (2).

RQD (%) = 110–2.5Jv ……………………… ***(2)***.

where RQD = 0 for Jv > 44 and RQD = 100 for Jv < 4. Random joints are included because they represent a significant part of the number of measured discontinuities, neglecting them would lead to erroneous quantifications of the discontinuity nature of rock mass (Grenon and Hadjigeorgiou, 2003). As suggested by Palmström (1982), the spacing (S) of 5m for each random joint was taken, thus, the volumetric joint count (Jv) can be calculated from the Equation (3):

$$\mathrm{Jv} = \frac{1}{S1} + \frac{1}{S2} + \frac{1}{S3} + \frac{1}{Sn} + \frac{Nr}{5\sqrt{A}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \ (\mathbf{3}).$$

where, S1, S2 and S3 are the average spacing's for joint sets, Nr is the number of random joints in the actual location and A is the area in m².

The orientations (dip/dip direction (deg.)) of joints and bedding planes as well as their characteristics and conditions viz; persistence (m), aperture (mm), roughness, state and thickness of filling material, water flow and wall weathering were measured/estimated in the field according to the procedures recommended by ISRM (1981).

Based on the value of RMR, the rock mass could be plotted into five classes: very good (RMR 100–81), good (80–61), fair (60–41), poor (40–21), and very poor (<20).

3.2 Rock Slope Kinematic Analysis (RSKA)

Kinematic analysis is based on the Markland Test Plot method as described by Hoek and Bray (1981), developed by Goodman (1989) and modified by Wyllie and Mah (2004). In this study 12 rock cut slope locations were assessed using SMR system and then were further investigated through kinematic analysis. The orientations (dip and dip direction) of the main discontinuity (joint /bedding plane) sets (βj / αj) and orientation of each slope face (βs/αs) obtained from the field measurements by employing the geological compass were used in the calculation of SMR (Table 3) and the same were used to perform kinematic analysis to identify the probable failure modes (plane, wedge, toppling failures) due to unfavorably oriented discontinuities within the slope-forming rock mass. Kinematic analysis was conducted by re-plotting the orientations (βj and αj) of the recognized main joint/bedding sets and orientation of each slope face (βs/αs) for each rock slope location employing stereo-net software, version 8(Allmendinger, 2013). The internal friction angle (Ø°) of each rock mass used for kinematic analysis has been estimated based on the RMR values.

3.3 Determination of the original Slope Mass Rating (SMR)

The stability of slope was assessed by the original slope mass rating (SMR) system proposed by Romana (1985) as an application of Bieniawski (1989) basic Rock Mass Rating system ($RMR_{basic-89}$).

The SMR is originally derived from the $RMR_{basic-89}$ by adding "correction factors" (F_1, F_2 and F_3) derived from joint–slope relationship and factor depending on the excavation method or nature of slope (F_4) (Table 2), as shown in Equation (***4***):

$SMR = RMR_{basic-89} + (F_1 \times F_2 \times F_3) + F_4$**………………………** ***(4)***.

Where:$RMR_{basic-89}$ is calculated as mentioned above; F_1 is an correction factor that represents the difference between the dip direction of a discontinuity (α_j) (or the plunge direction of the intersection line of two planes (α_i)) and the dip direction of a rock slope face (α_s) with a value of 0.15 to 1.0; F_2 refers to dip angle of joint (β_j) in the planar mode of failure or the plunge of the intersection line of two planes (βi) in the wedge mode of failure; F_3 states the relationship between the slope face dip (βs) and joint dips (βs) or the plunge of the intersection line of two planes (βi); F_4 the correction factor that depends on the method of excavation (Table3).

On the basis of the SMR values, the stability of rock slopes is classified as shown in Table 4 into five main classes. These are: I (Completely Stable) (when SMR value=81-100), II (Stable) (when SMR value=61-80), III (Normal) (when SMR value=41-60), IV (Unstable) (when SMR value= 21-40), and V (Completely Unstable) (when SMR value=0-20). Mode and probability of failures were also inferred from the SMR Values (see Table 4).

Table 2: Correction parameters for SMR (modified from Romana, 1985 by Anbalagan et al., 1992)

Type of failure			Very favourable	Favourable	Normal	Unfavourable	Very unfavourable
P	A=	|αj-αs|	> 30°	30-20°	20-10°	10-5°	< 5°
W		|αi-αs|					
T		|αj-αs-180|					
P/W/T		F1	0.15	0.40	0.70	0.85	1.00
P	B =	| βj |	< 20°	20°-30°	30°-35°	35°- 45°	> 45°
W	B =	| βi|					
P/W		F2	0.15	0.40	0.70	0.85	1.00
T		F2	1.00	1.00	1.00	1.00	1.00
P	C =	Bj -βs	>10°	10°- 0°	0°	0°-(-10°)	<(-10°)
W		Bi -βs					
T		Bj +βs	<110°	110°-120°	>120°	----	----
P/W/T		F3	0	-6	-25	-50	- 60

P: Planar failure; T: Toppling failure; W: Wedge failure; α j: Dip direction of discontinuity; αi: Plunge direction of line of intersection two discontinuities; αs: slope; βj: Dip of discontinuity; βi: Plunge of line of intersection two discontinuities; βs: Dip of slope.

Table 3: Correction ratings for methods of excavation of slopes (after Romana, 1985)

Excavation Method	Natural slope	Pre-splitting	Smooth blasting	Normal blasting/ Mechanical	Deficient blasting

F4	+15	+10	+8	0	−8

The appropriate mitigation measures were proposed for each classes based on the field guidelines and recommendations provided in SMR system and on the field observations in order to minimize any future impact of the slope failure.

Table 4: Description of SMR classes (after Romana, 1985)

Class No.	V		IV		III		II		I	
	Vb	Va	IVb	IVa	IIIb	IIIa	IIb	IIa	Ib	Ia
SMR Value	1-10	11-20	21-30	31-40	41-50	51-60	61-70	71-80	81-91	91-100
	0 - 20		21 - 40		41 - 60		61 - 80		81 - 100	
Description	Very bad		Bad		Normal		Good		Very Good	
Stability	Completely Unstable		Unstable		Partially Stable		Stable		Completely Stable	
Failure	Big Planner or Soil like		Planer or Big Wedges		Some joints or many Wedges		Some Blocks		None	
Failure Probability	0.9		0.6		0.4		0.2		0	

3.4 Determination of theLandslide Possibility Index (LPI)

The computation of the Landslide Possibility Index (LPI) aids in the determination of the degree of the hazard and is based upon 10 main characteristic features namely: slope height, slope angle, grade of fracture of the rock mass, grade of weathering of the rock mass, gradient of the discontinuities, spacing of the discontinuities, orientation of the discontinuities, vegetation cover, water infiltration and previous landslides (Table 5). These parameters were estimated and quantified in the field for each slope following the procedure recommended by Bejerman (1994). Each parameter has a range of values (Table 5). Estimated values of 10 parameters are added to determine the Landslide Possibility Index (LPI) value for the slope as shown in the following equation:

LPI= $\sum_{i=1}^{10}(\text{estimated value s}) = 1 + 2 + 3 + 4 \pm 5 + 6 + 7 + 8 + 9 + 10$ …………… (5).

If the orientation of the discontinuities is favorable, the estimated value of the discontinuity gradient is subtracted from the total LPI for the purpose of correcting the LPI value (Bejerman, 1994). Based on the obtained value of LPI, the possibility of failure of slopes were classified into six categories (S, VL, L, M, H and VH), while the degree of landslide hazard was grouped into three classes (L, M and H) (Table 6).If the value of LPI is greater than 21, slope is highly hazardous; LPI value of 11-20 means slope is moderately hazardous; and if LPI value is less than 10, slope plots in low hazard area (Table 6).

Table 5: Work sheet for Landslide Possibility Index (LPI) Calculation (after Bejerman, 1994)

NUMBER SLOPE ☐☐☐ PAGE ☐☐☐

KILOMETER ☐☐☐☐☐ DATE ☐☐☐☐☐☐

1. SLOPE ESTIMATE HEIGHT (m)		2. SLOPE ESTIMATE ANGLE		3. GRADE OF ESTIMATE FRACTURE	
1 ----------- 8	1	< 15°	0	Sound	0
9 ----------- 15	2	15° ------ 30°	1	moderately fractured	1
16 ---------- 25	3	30° ------ 45°	2	highly fractured	2
26 ---------- 35	4	45° ------ 60°	3	completely fractured	3
> ---------- 35	5	> 60°	4		

4. GRADE OF ESTIMATE WEATHERING		5. GRADIENT OF ESTIMATE THE DISCONTINUITIES		6. SPACING OF ESTIMATE THE DISCONTINUITIES	
Fresh	0	< 15°	0	> 3 m	0
slightly weathered	1	15° ------ 30°	1	1 --------- 3 m	1
moderately weathered	2	30° ------ 45°	2	0.3 ------- 1 m	2
highly weathered	3	45° ------ 60°	3	0.05 --- 0.3 m	3
completely weathered	4	> 60°	4	< 0.05 m	4
residual soil	5				

7. ORIENTATION OF THE ESTIMATE DISCONTINUITIES		8. VEGETATION ESTIMATE COVER	
favorable	0	void (< 20%)	0
unfavorable	4	scarce (20 - 60%)	1
		abundant (> 60%)	2

9. WATER INFILTRATION ESTIMATE		10. PREVIOUS LANDSLIDES ESTIMATE	
inexistent	0	Not registered	0
scarce	1	registered (small volume)	1
abundant: permanent	2	registered (high volume)	2
abundant: seasonal	3		

1 + 2 + 3 + 4 ± 5 + 6 + 7 + 8 + 9 + 10 = ☐☐

I (small) (0 – 5) III (low) (11 – 15) V (high) (21 – 25) ☐☐☐
II (very low) (6 – 10) IV (moderate) (16 – 20) VI (very high) (> 25)

The LPI value is obtained by adding the estimations of attributes 1 to 10. If the orientation of the discontinuities is favorable, subtract the estimation of gradient.

OBSERVATIONS:

Table 6: Landslides Hazard Index categories (After Bejerman, 1994; 1998)

Landslide Possibility Index (LPI)			Landslide Hazard Category
LPI	LPI Category	Failure Possibility	
(0-5)	I	Small; S	Low Hazard; L
(6-10)	II	Very Low; VL	
(11-15)	III	Low; L	Moderate Hazard; M
(16-20)	IV	Moderate ; M	
(21-25)	V	High ; H	High Hazard; H
(>25)	VI	Very High; VH	

4. RESULTS AND DISCUSSION

4.1 Slope stability and Hazard evaluation of rock cut slope locations

The results of the measured/estimated values of the RMR, RSKA, SMRand LPI parameters at 12 rock cut slopes along Al-Husiah road following the standard procedures are presented in Table 7.The computed values of RQD index based on the valuesof Jv (j/m³) are shown in Table 8.

Table 7: Field data obtained at12 rock cut slopes along Al-Husiah road.

Slope location no.		1	2	3	4	5	6	Remarks
Slope Face orientation (βsf° /αsf °)		70/200	75/230	80/210	40/250	85/220	85/210	
Slope height (m)		25	15	8	14	19	12	
Vegetation cover		Scarce (20 - 60 %)		Void (< 20 %)			Scarce	
Water infiltration		Abundant: seasonal						
Previous landslides		Registered small volume						
	Set1 (J0)	30/040	40/035	35/075	45/050	45/210	45/040	
9		(0.32)	(0.47)	(0.29)	(0.41)	(0.39)	(0.24)	
70/225	Set1 (J1)	70/240	45/260	35/190	50/210	45/240	40/075	
		(0.40)	(0.40)	(0.10)	(0.20)	(0.20)	(0.10)	From field
8	Set2 (J2)	55/185	60/190	45/245	50/330	40/140	45/221	
- 60 %)		(0.70)	(0.70)	(0.30)	(0.22)	(0.70)	(0.30)	
Abundant: sea	Set3 (J3)	20/100	40/100	30/320	60/100	20/200	10/120	
Registered small		(0.30)	(0.30)	(0.90)	(0.33)	(0.60)	(0.90)	
45/040	Set4 (J4)	-	-	-	-	-	-	
(0.65)		-	-	-	-	-	-	
50/290	Set 5 (random)	5/4=1.25	5/3=1.67	5/5=1	5/5=1	5/4=1.25	5/5=1	
(0.10)	Min. Spacing	0.30	0.30	0.10	0.20	0.20	0.10	
Ground water condition		C.dry	C.dry	C.dry	C. dry	C. dry	C.dry	
[Rating]		15	15	15	15	15	15	
Discontinuities Condition and ratings (as average)	Persistence (m)	<1-3	<1-3	<1-3	<1-10	<1-3	<1-10	
	[Rating]	[5]	[5]	[5]	[4]	[5]	[4]	
	Aperture (mm)	1-5	>5	1-3	>5	>5	>5	From field and Bieniawskis' Table , section (E)
	[Rating]	[1]	[0]	[1]	[0]	[0]	[0]	
	Roughness	Rough	Rough	Rough	Smooth	Smooth	Slickensided	
	[Rating]	[5]	[5]	[5]	[1]	[1]	[0]	
	Infilling	Sf.<5mm	Sf. > 5mm	Sf.<5mm	Sf. > 5mm	Sf. >5mm	Sf. >5mm	
	[Rating]	[2]	[0]	[2]	[0]	[0]	[0]	
	Weathering	Slightly	Md	Md	Md-H	Md-VH	Sly-H	
	[Rating]	[5]	[3]	[3]	[2]	[1]	[3]	

Table 8: Values of Rock Quality Designation (RQD, %) index calculated based on values of Jv (j/m³)

Slope location no.	1	2	3	4	5	6	7	8	9	10	11	12	Remarks
Jv (j/m³)	11.19	9.99	18.89	16.02	11.46	19.61	17.06	12.64	16.58	10.79	29.14	26.64	From Table 7 and Eq. 3.3
RQD %	82	85	62.77	69.96	81.45	60.97	67.36	78.39	68.54	83.02	37.14	43.39	From Eq. 3.2
Where: Jv (j/m³) = volumetric joint count													

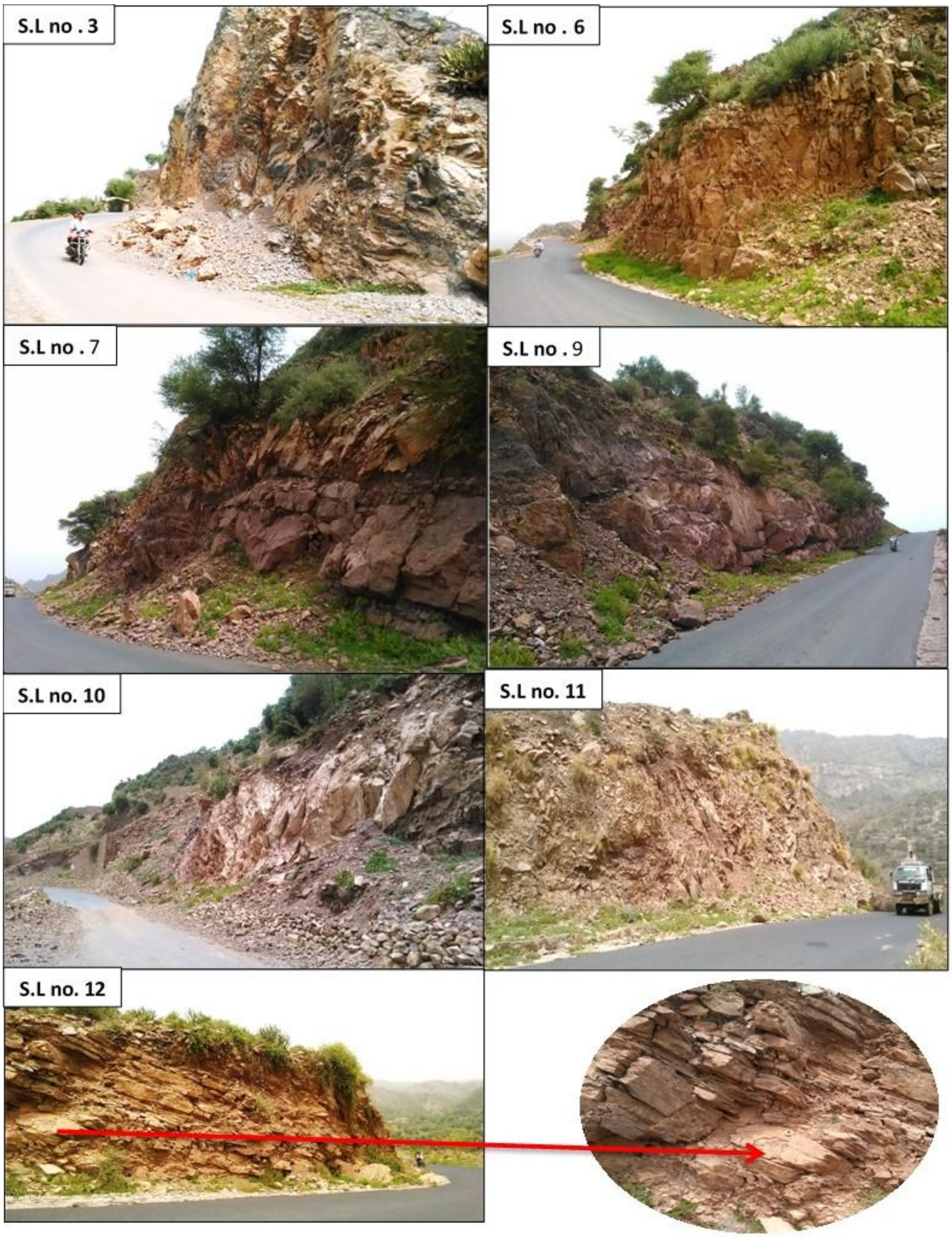

Figure 5: Field photographs showing fewrock cut slopes along Al-Husiah road

4. 1.1. Evaluation of RMR values

The RMR$_{basic-89}$five parameters were measured in the field at 12 rock cut slopes as per the procedure proposed by Bieniawski (1989) in the RMR system. The average values of five input parameters, their ratings and the final basic RMR$_{-89}$values for the rock mass of eachcut slope location are shown in Table 9. From the Table 9, it is evident that the average values of RMR$_{basic-89}$ for each cut slope show significant difference; the rock quality rating varies from37.5 to 73.5. The values of good rock (class II) range from 62 to 73.5, fair rock from 42 to 56 (class III) and the value at location No. 12 is 37.5, thus considered as poor rock (class IV). The rock mass with good quality is encountered at the slope location nos. 1, 2, 8 and 10, while the slope location nos. 3, 4, 5, 6, 7, 9 and11are characterized by fair quality rock mass. Structurally, the rock masses at all these locations have been subjected to faulting due to subsequent tectonic events in the western part of Yemen. Variation in the RMR values of the rock masses along the cut slopes may be attributed to differing lithology, structure and weathering condition.

4. 1.2. Rock Kinematic Analysis

Internal friction angles of rock acquired from the final basic RMR rating were used to perform kinematic analysis for 12 rock slopes along the Al-Husiah route (see Table 9). The orientations (dip and dip direction) of the main discontinuity (joint / bedding plane) (j / j) sets and the orientation of each slope face (s/s) obtained from field measurements (see Table 7) were plotted in stereonet to perform kinematic analysis, which was proposed to identify the possible failure type (plane, wedge, toppling failures) within the slope-forming rock mass (Fig. 6), and the same data were used in the calculation of SMR.Accordingly, the kinematic analysis in slope nos. 1, 2, 3, 4, 8, 9 and 12 reveals that the intersection between J1 and J2 form the wedge-shaped block failure. Also, the most unfavorable condition for the wedge-shaped block failure is the result of intersection formed by the beddings Jo and J1 as in slope nos. 5, 8 and 12 or Jo and J2 as in slope no. 8. In both mentioned cases, the plunge angles (βi) of the line of intersection develop either among J1 and J2 or among Jo and J1 or J2 are higher than internal friction angle (Ø°) and lower than dip angles of that slopes "daylighting".

As a result, kinematic analysis of slopes 1, 2, 3, 4, 8, 9, and 12 reveals that the junction of J1 and J2 results in a wedge-shaped block failure. Also, the junction created by the beddings Jo and J1 as in slopes 5, 8, and 12, or Jo and J2 as in slopes 8, is the most unfavourable circumstance for wedge-shaped block failure. The plunge angles I of the line of intersection that emerge between J1 and J2 or between Jo and J1 or J2 are larger than the internal friction angle (Ø°) and lower than the dip angles of the slopes "daylighting" in both situations.

Table 9 The five input parameter values and their ratings required in the calculation of the basic RMR_{-89} for the rock mass of the slope locations along the studied road

Slope L. No.	Lithology	UCS		RQD		Spacing (DS)		Condition of disc (DC)		G.W condition (GWC)		$RMR_{basic-89} = R_{ucs}+R_{RQD}+R_{DS}+R_{DC}+R_{GWC}$			Ø (deg.)
		Values	Rating (R_{UCS})	Values	Rating (R_{RQD})	Values (m)(min.)	Rating (R_{DS})	Values	Rating (R_{DC})	Values (disc.)	Rating (R_{GWC})	Rating	RMC	RMD	
1	Sandstone - Shale/ Mds	33	4	82	17	0.30	10	Discontinuities Condition (as average)	18	C. dry	15	64	II	Good	36.58
2	Sandstone - Shale/ Mds	63	7	85	17	0.30	10		13	C. dry	15	62	II	Good	36.53
3	Sandstone	33	4	62.77	13	0.10	8		16	C. dry	15	56	III	Fair	32.89
4	Sandstone - Shale/ Mds	28	4	69.96	13	0.20	10		7	C. dry	15	49	III	Fair	29.21
5	Sandstone - Shale/ Mds	28	4	81.45	17	0.20	10		7	C. dry	15	53	III	Fair	31.31
6	Sandstone	75	7	60.97	13	0.10	8		7	C. dry	15	50	III	Fair	29.73
7	Sandstone - Shale/ Mds	75	7	67.36	13	0.10	8		5	C. dry	15	48	III	Fair	28.68
8	Sandstone - Shale/ Mds	150	12	78.39	17	0.20	10		19.5	C. dry	15	73.5	II	Good	41.58
9	Sandstone - Shale/ Mds	75	7	68.54	13	0.10	8		4	C. dry	15	47	III	Fair	28.16
10	Sandstone - Shale/ Mds	33	4	83.02	17	0.21	10		17.5	C. dry	15	63.5	II	Good	36.32
11	Sandstone	26	4	37.14	8	0.12	8		7	C. dry	15	42	III	Fair	25.61
12	Shale/ Mudstone	13	2	43.39	8	0.08	8		4.5	C. dry	15	37.5	IV	Poor	23.44
Remarks		From field (as ave.)	Bieniawskis' Table, section (A1)	From Table 8	Bieniawskis' Table, section (A2)	From Table 7	Bieniawskis' Table, section (A3)	From Table 7	Bieniawskis' Table, section (E)	From Table 7	Bieniawskis' Table, section (A5)	Based Eq.(3.1)	Bieniawskis' Table , section (C)		Bieniawskis' Table , section (D)

Where: L.: location, $RMR_{basic-89}$ = Basic RMR_{-89} with no adjusting factor for joint orientation, R_{ucs}: ratings for the uniaxial compressive strength of the intact material (UCS; MPa), R_{RQD}: ratings for the Rock Quality Designation (RQD %), R_{DS}: ratings for the spacing of discontinuities (minimum spacing, according to Edelbro, 2003), R_{DC} :ratings for the condition of discontinuities, R_{GWC} : ratings for the groundwater condition, G.W: Groundwater, C.dry: Completely dry, (disc.): descriptive term, RMC: Rock mass class, RMD: Rock mass description according to Bieniawski (1989), Muds: Mudstone, Ø (deg.):Friction angle of rock mass in degree.

Also, the intersection between J1 and J2 in slopes 7 and 10 trigger the wedge failure, but the difference between dip directions of line of intersection (αi) of those joints and dip directions of slopes (αs) is more than 20 ^{0} , so no potential wedge failure can exist, while the wedge failure is impossible along the intersection line of the bedding Jo and J2 in slope 10 and Jo and J4 in slope 11 due to the plunge angle of the intersection line (βi) develop among those joint / bedding sets are lower than internal friction angle (Ø°) (Fig. 6). Thewedge failure type is the dominant one among the three failure modes in the investigated slope locations (42%).

As shown in Figure 6; the joint sets J1 (slope nos. 3, 5 and 7), J2 (slope nos. 1, 6, 11 and 12) and bedding Jo (slope nos. 5 and 8) represent the most unfavorable conditions for planar /rockfall failures due to their orientations and conditions. In other words, the following conditions of a kinematic plane failure are almost met: 1) βs > βj > Ø° and 2) |αj- αs| < 20°.The planar /rockfall failures along J1 in slope 2 and along bedding Jo in slope 10 are not expected because those slopes and dip angles of that planes are not parallel i.e. |αj- αs| > 20°, while in slope 5 the planar /rockfall failure along J3 set is impossible due to the dip angle of the j3 is less than the internal friction angle (Ø°).

According to kinematic analysis, the toppling along the bedding Jo (toppling/rockfall controlled by bedding) in slope location nos. 2, 4, 6, 7 and 9 is kinematically possible to be occurred; because the required conditions for toppling failure along Jo plane are met.The toppling /rockfall failures along Jo in slope 3 and along J3 in slopes 4 and 12 are not expected because those slopes and dip angles of that planes are not parallel i.e. $|\alpha j- \alpha s| > 20°$.However, those cut slopes are conditioned mainly by almost vertical slopes of fractured and weathered sandstone rock mass, thereby favoring rockfall as seen in the field.

4. 1.3. Evaluation of SMR values

SMR system was adapted to assess slope stability at 12 rock cut slope locations after RMR basic-89 and kinematic analyses.SMR iscomputed from $RMR_{basic-89}$by determining “correction factors” (F1, F2 and F3) derived from joint–slope relationship and factor depending on the excavation method or nature of slope (F4) as shown in Eq. (4).The relationship between the orientations (dip and dip direction) of the main discontinuity (joint /bedding plane) (βj / αj) sets and orientation of each slope face (βs/αs) was kinematically determined and the probable failure modes were identified (see Table 10 and Fig. 6).The

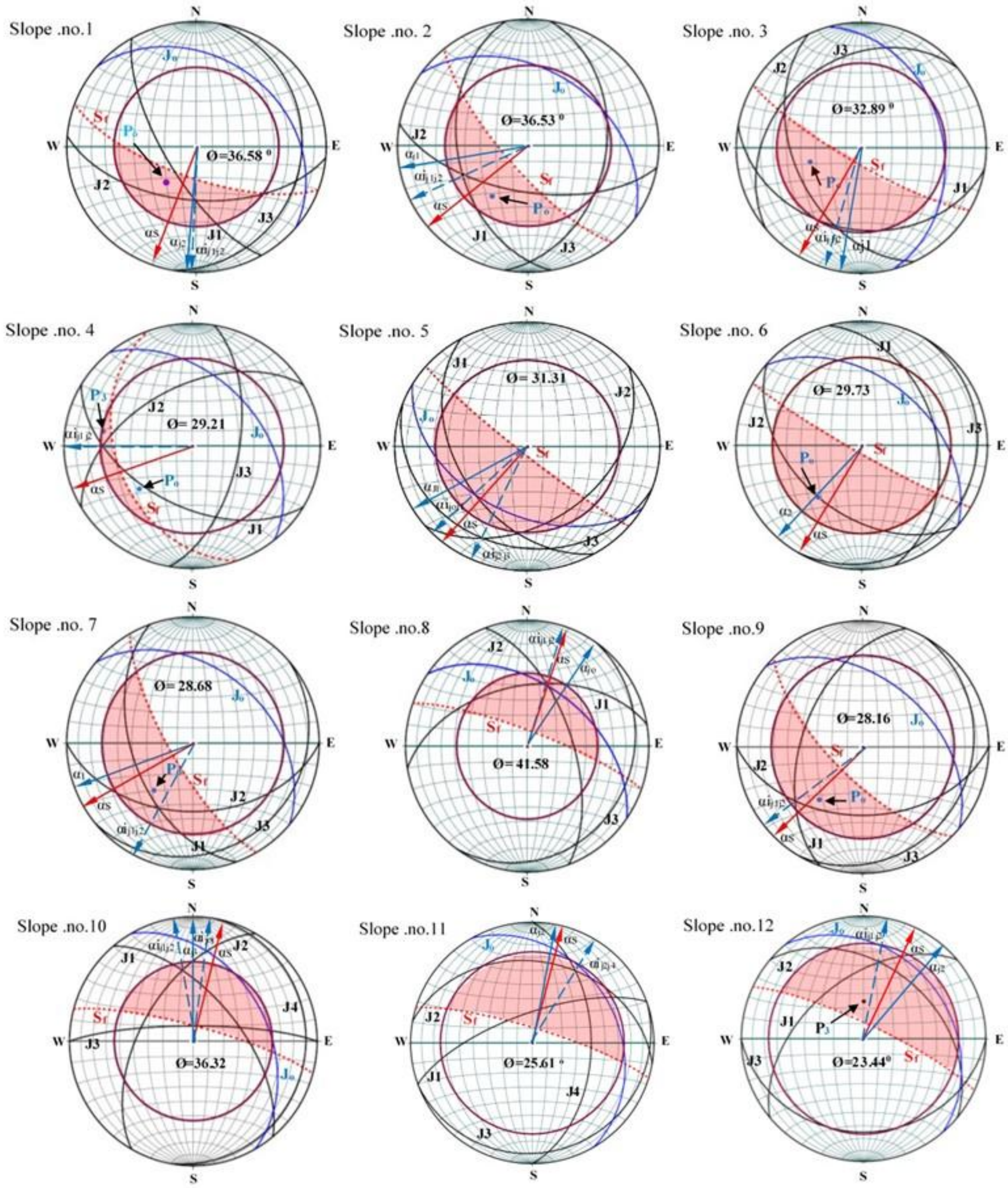

Figure 6: Plot of main joint/bedding sets and slope faces for 12 rock cut slope locations. The pink coloured area indicates the critical zone of failure. The symbols used in the figure are: J1=Joint set1, J2= Joint set2,…and J4= Joint set4; Jo= bedding; α_s: dip direction of slope

Obtained results of SMR for 12 rock cute slope location are listed in the Table 11. The values of SMR show variations from 1.8 to 64 indicating that these values plot from "Very Bad" class (Vb) to "Good" class (IIb). The ranges of SMR values for rock cut slopes characterized by very bad and completely unstable conditions (Class No. V) against planner failure (slope locations 5, 6, 7,11 and 12) and wedge failure (slope locations 4, 8, 9 and 12) are 1.8 - 17.3 and 1.8 - 19.6 respectively. This reveals that the failure probability of these slopes is 90%. The results also indicate that the SMR values of slopes 1, 3,6, 8 and 12 are 22, 39.2, 28.75, 31.5 and 37.5respectively and are classified as "Bad" and are in unstable conditions (Class no. IV) against planner and toppling failures and the failure probability for all these slopes is 60%. The slopes 2, 3, 4, 7 and 9 have SMR values of 57.2, 55.1, 49, 43.8 and 41.9 respectively, indicating that these values plot under "Normal" (III) class and these rock slopes are in partially stable conditions against toppling failure, and the probability of failure is 40 %. The various stabilities and modes of failure in the studied rock cut slopes along Cretaceous sandstone sequence are shown diagrammatically in Figure 7.The results also indicate the suitable corrective measures must be taken to control slope failures, especially in rock cut slopes where the SMR values fall in IV and V classes.Based on values of SMR system and field observations the remedial measures to control slope failures at 12 slope locations were suggested as shown in the Table 11.

4. 1.4. Evaluation of LPI value

The Landslide Possibility Index (LPI) suggested by Bejerman (1994, 1998) was adopted to assess the degree of hazard in 12 road cut slope location. The ten parameters of LPI-system were estimated in the field for each location and the final results are shown in Table 12.The obtained results of LPI values indicate that the possibility of failure and the degree of hazard in 10 rock cut slope locations namely 1, 2, 3, 4, 5, 6, 7, 8, 9 and 10 are "High", while in the remained rock cut slope locations viz., 11 and 12, the degree of hazard are "Very High" and"“High" respectively.

Table 11: Results of slope mass rating (SMR) for 12 rock cut slope location along the studied road

Slope location No.	Critical condition	Probable failure mode	RMR_{8-89}	F1	F2	F3	F4	SMR Value	Class No.	Description	Stability	Probability type of failure	Failure Probability %	Support measures	
														SMR ave.	Field Obs.
1	Jo	T/F	64	0.70	1	0	0	64	IIb	Good	Stable	Some Blocks	20	1	a
	J1& J2	W		0.70	1	-60	0	22	IVb	Bad	Unstable	Planar or Big Wedges	60		
	J2	P		0.70	1	-60	0	22	IVb	Bad	Unstable	Planar or Big Wedges	60		
2	Jo	T/F	62	0.70	1	-6	0	57.2	IIIa	Normal	Partially Stable	Some joints or many Wedges	40	2	a
	J1& J2	W		0.70	0.85	-60	0	26.3	IVb	Bad	Unstable	Planar or Big Wedges	60		
3	Jo	T/F	56	0.15	1	-6	0	55.1	IIIa	Normal	Partially Stable	Some joints or many Wedges	40	1	a
	J1	P		0.40	0.70	-60	0	39.2	IVa	Bad	Unstable	Planar or Big Wedges	60		
	J1& J2	W		0.70	0.70	-60	0	26.6	IVb	Bad	Unstable	Planar or Big Wedges	60		
4	Jo	T/F	49	0.70	1	0	0	49	IIIb	Normal	Partially Stable	Some joints or many Wedges	40	1	b
	J3	T		0.40	1	0	0	49	IIIb	Normal	Partially Stable	Some joints or many Wedges	40		
	J1& J2	W		0.70	0.70	-60	0	19.6	Va	Very Bad	Completely Unstable	Big Planner or Soil like	90		
5	Jo	P	53	0.85	0.85	-60	0	9.65	Vb	Very Bad	Completely Unstable	Big Planner or Soil like	90	3	b
	J1	P		0.70	0.85	-60	0	17.3	Va	Very Bad	Completely Unstable	Big Planner or Soil like	90		
	Jo& J2	W		0.85	0.85	-60	0	9.65	Vb	Very Bad	Completely Unstable	Big Planner or Soil like	90		
	J2 & J3	W		0.70	0.15	-60	0	46.7	IIIb	Normal	Partially Stable	Some joints or many Wedges	40		
6	Jo	T/F	50	0.85	1	-25	0	28.75	IVb	Bad	Unstable	Planar or Big Wedges	60	3	a
	J2	P		0.70	0.85	-60	0	14.30	Va	Very Bad	Completely Unstable	Big Planner or Soil like	90		
7	Jo	T/F	48	0.70	1	-6	0	43.8	IIIb	Normal	Partially Stable	Some joints or many Wedges	40	3	b
	J1	P/F		0.70	1	-60	0	6	Vb	Very Bad	Completely Unstable	Big Planner or Soil like	90		
8	Jo	P	73.5	0.70	1	-60	0	31.5	IVa	Bad	Unstable	Planar or Big Wedges	60	4	a
	Jo & J1	W		0.85	1	-60	0	22.5	IVb	Bad	Unstable	Planar or Big Wedges	60		
	Jo & J2	W		1	1	-60	0	13.5	Va	Very Bad	Completely Unstable	Big Planner or Soil like	90		
	J1& J2	W		1	1	-60	0	13.5	Va	Very Bad	Completely Unstable	Big Planner or Soil like	90		
9	Jo	T/F	47	0.85	1	-6	0	41.9	IIIb	Normal	Partially Stable	Some joints or many Wedges	40	3	a
	J1& J2	W		0.85	0.70	-60	0	11.3	Va	Very Bad	Completely Unstable	Big Planner or Soil like	0.9		
10	J3	P/F	63.5	0.70	1	-25	0	46	IIIb	Normal	Partially Stable	Some joints or many Wedges	40	2	a
11	J2	P	42	0.85	0.70	-60	0	6.3	Vb	Very Bad	Completely Unstable	Big Planner or Soil like	90	4	c
	J2 & J4	W		0.70	0.70	-60	0	12.6	Va	Very Bad	Completely Unstable	Big Planner or Soil like	90		
12	J2	P/F	37.5	0.70	0.85	-60	0	1.8	Vb	Very Bad	Completely Unstable	Big Planner or Soil like	90	4	c
	J3	T		0.40	1	0	0	37.5	IVa	Bad	Unstable	Planar or Big Wedges	60		
	Jo & J1	W		1	0.40	-60	0	13.5	Va	Very Bad	Completely Unstable	Big Planner or Soil like	90		
	J1& J2	W		0.70	0.85	-60	0	1.8	Vb	Very Bad	Completely Unstable	Big Planner or Soil like	90		
Remarks	From Fig, 6 and Table 2		From Table 9	From Table 2			Table 3	Eq.4	From Table 4					Romana, 1993	Field

Where: SMR ave., and Field obs.: the support measures have been suggested based on the average SMR and Field observations; (1): Anchors Systematic shotcrete, Toe wall and/or concrete, (Reexcavation) Drainage; (2): (Toe ditch and/or nets), Systematic bolting. Anchors Systematic shotcrete, Toe wall and/or dental concrete; (3): Systematic reinforced shotcrete, Toe wall and/or concrete, Reexcavation, Deep drainage; (4):Gravity or anchored wall, Reexcavation; (a): mechanical removal of failing blocks, slide debris from the roadsides and potentially unstable rock blocks from upper part of the slope and erection of control works in the form of U- ditches at slope toe long both sides of the road to collect the detached and fallen rock fragments and surface water draining; .(b): construction of some retaining walls; .(c): re-excavation.

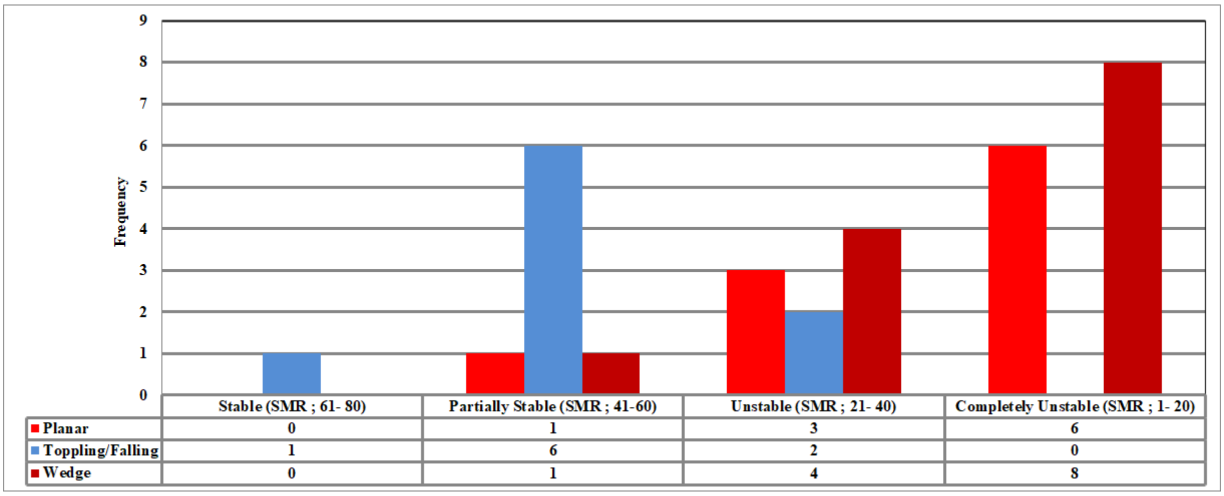

Figure7: Type of failures and stability classes of the rock cut slopes along Al-Husiah road, Yemen

Table 12:LPI rating and degree of the hazard at 12 rock cut slopes

No.	Characteristic features of the slope	Rock cut slope location no.												Remarks
		1	2	3	4	5	6	7	8	9	10	11	12	
1	Slope Height	3	2	1	2	3	2	2	5	1	3	2	2	From Table 7 and Table 5
2	Slope angle	4	4	4	2	4	4	4	4	4	4	4	4	
3	Grade of fracture of the rock mass	1	1	2	2	1.5	2.5	2.5	0	1.5	2	3	3	
4	Grade of weathering of the rock mass	1	2	2	2.5	2.5	2	3	0.5	3	2	3.5	3.5	
5	Gradient of the discontinuities	2	2	2	2	2	2	2	2	2	2	3	2	
6	Spacing of the discontinuities	2	2	2	3	2	3	2	2	2	2	3	3	
7	Orientation of the discontinuities	4	4	4	4	4	4	4	4	4	4	4	4	
8	Vegetation cover	1	1	0	0	0	1	1	1	1	1	0	0	
9	Water infiltration	3	3	3	3	3	3	3	3	3	3	3	3	
10	Previous landslides	1	1	1	1	1	1	1	1	1	1	1	1	
Landslide Possibility Index (LPI) = Σ estimations (1-10)		22	22	21	21.5	23	24.5	24.5	22.5	22.5	24	26.5	25.5	Eq. 5
LPI category		V	V	V	V	V	V	V	V	V	V	VI	VI	From Table 6
Failure possibility		H	H	H	H	H	H	H	H	H	H	V.H	V.H	
Hazard category		H	H	H	H	H	H	H	H	H	H	H	H	

4.2 Causes of the landslides

Detailed field studies carried out along Al-Husiah road, Yemen by the present authors brought to light the following factors which are considered to beresponsible for most of the landslides or slope failures in road cut exposures:

1. Most of the landslides have occurred during rainfall periods which start from April to June with peak in May and from August to October with peak in September. From the compiled data starting from the year 1979 till date, it is known that the study area has received anaverage annual rainfall of about 520 mm. This amount of rainfall acted as a triggering factor for causing instability at the road cut slopes and consequently rendering them as areas of 'high risk' for rock/soil slides and rockfalls.

2. In most of the cut slopes, the rock mass is composed of jointed sandstone alternating or intercalated with varicolored, weathered, weakly welded shale/mudstone rock layers. The presence of weak shale/mudstone layers within the sandstone layers probably facilitated the sliding and made some of the slopes dangerous and potential sites for slope failure.

3. In the investigated slopes, sliding frequently takes place along persistent planar

discontinuities such as bedding planes and/or joint surfaces (Fig. 4).

4. Overhanging of the jointed sandstone layers which often have discontinuities with unfavorable orientation,

5. In the study area, the natural slopes have been modified by construction of the road and/or by the extraction of rock mass for construction purposes (quarrying activities). These modifications have reduced the slope stability and the slopeshave become unstable and are prone to failure, especially during rainfall periods.

6. In addition, dip of the some sandstone sequence layers is towards the road and the bedding planes are daylighting in the cut slope face (bedding plane dip < slope dip), so they are susceptible to different types of mass movements (sliding, toppling, falling, etc.).

7. Poor drainage conditions of slope: There is no drainage at top or toe of slope.

5. CONCLUSIONS

In this study, the slope stability at 12 rock cut sites along Al-Husiah road was assessed through integrated methods in which several geomechanical parameters were considered and measured/estimated in the field. The procedure allowed for characterization of rock mass (through RMR), identification of the probable failure mode (kinematic analysis; RKA), stability assessment of the cut slopes (through SMR) and determination of the degree of the hazard of the slopes using LPI system.Furthermore, the various factors responsible for the slope failures in the study area were also identified. The following conclusions are drawn based on the integrated studies:

1. The rock cut slopes along Al-Husiah road have been subjected to various types of failures and a single slope may have been affected by more than one type of failure depending on the joint/bedding - slope face relationship, joint/bedding characteristics, lithological conditions and rainfall.

2. The rock mass quality assessed by basic RMR_{-89}system in all the cut rock slopes was varied between good (class no. II) and poor (class no. IV) rock quality.The RMR average values for good, fair and poor quality were 65.8, 49.3 and 37.5 respectively.

3. The values of SMR also show variations from 1.8 to 64 indicating that these values correspond to "Very Bad" class (Vb) to "Good" class (IIb). This variation is attributed to various ratings of the basic RMR_{-89} and the inter-relationship between the joint/bedding and slope face.

4. The ranges of obtained SMR values at rock cut slopes categorized into 'Very Bad' and completely 'Unstable' conditions (class no. V) against planner failure (at site Nos. 5, 6, 7,11 and 12) and wedge failure (at site Nos. 4, 8, 9, 11 and 12) are 1.8 - 17.3 and 1.8 - 19.6 respectively and the failure probability of these slopes is 90 %. Only one slope (at site No. 1) is 'Stable' and remaining six rock slopes (at site Nos. 2, 4, 7 and 9) are 'Partially Stable' against toppling failures. The stability of three slopes (at site Nos. 1, 3& 8), two slopes (at site Nos. 6 & 12) and four slopes (at site Nos.1, 2,3& 8) fall under unstableclass (IV) and in bad conditions against planner, toppling/falling and wedge failure respectively. The failure probability of these slopes is 60 %.

5. Based on the average SMR value, three slope locations (8, 11, 12) are classified as very bad (class no. V), eight (1, 3,4 ,5 ,6 ,7, 9) as bad (class no. IV) and two (2, 10) as normal (class no. III).

6. The wedge failure is the dominant one among the three failure modes in the investigated slope locations (42%).

7. The results of SMR also indicate the suitable corrective measures must be taken to control slope failures, especially in cut rock slopes where the SMR values fall in V and IV classes. In the investigated cut slopes, the suggested remedial measures are mechanical removal of (i) failing blocks, (2) slide debris from the studied roadsides section and (3) potentially unstable rock blocks from upper part of the slope and (iv) erection of control works in the form of U- ditches at slope toe long both sides of the road to collect the detached and fallen rock fragments and (v) surface water draining during the heavy rainfall periods, especially at slope location nos. 1, 2, 3, 6, 8, 9 and 10. Furthermore, the construction of retaining walls to protect the slopes and to reduce the hazard is also required, especially at cut rock slope location nos. 4, 5, and 7. The re-excavation is proposed as corrective measure for the cut rock slopes nos. 11 and 12.

8. According to the application of the Landslide Possibility Index (LPI), it was found that the degree of landslide hazard of all investigated cut rock slopes is "high".

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