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KINEMATIC ANALYSIS OF SELECTED ROCK SLOPES ALONG CHOA SAIDAN SHAH-KALLAR KAHAR ROAD SECTION

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Abstract

The present study deals with the kinematic analysis of joint orientation data gathered from three selected slope sites situated along the Choa Saiden Shah-Kallar Kahar road section by using DIPS (5.1) software suite for Microsoft Windows. The stability analyses indicate that slope faces 1, 2 and 3 are not prone to planar and wedge failure modes; however, they carry toppling failure risk of 68%, 50% and 81% respectively. It is recommended that the selected slope sites may be stabilized by using rock anchors/bolts, proper drainage of water and the erection of rock catch fences etc.

Keywords: Rock Slope Stability, Kinematic Analysis, Limit Equilibrium Analysis, Equal Area Stereo-net, Pole Plot, Choa Saiden Shah-Kallar Kahar, Geological Discontinuities, Dips.

1. Introduction

Rock slopes exist either naturally or formed by man-made activities. Man made rock slopes are mainly created by surface mining excavations (open pits, stone quarries, open cast mines etc.) and by the construction of highways or motorways through mountainous regions. Even during underground mining operations the stacking of rock debris may pose a risk of slope failure and sometimes cause heavy damage to the mine surface plant. Failure of rock slopes both naturally occurring and man-made, include rock falls, rock or land-slides and overall slope instability. Rock slope failure is a catastrophic procedure of rock mass movement along inclined discontinuity planes and generally takes place due to instability of slopes. The consequence of such failures can range from direct costs such as loss of human life, damage to infrastructure (buildings, bridges, roads, other properties etc.) and vehicles, removal of the failed rock material, slope stabilization and the indirect costs including property depreciation, loss of production of industrial and agricultural sectors, reduction of tax income, traffic delays and enhanced maintenance costs [31].

Slope failures along roadside cuts or open pit mine walls are the effect of a variety of geologic and geometric parameters like engineering and lithologic properties of rocks (uniaxial compressive strength, slake durability of slope material, type and amount of clay minerals etc.),

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slope characteristics (slope angle, joint structure, surface runoff, ground water seepage etc.), rock mass characteristics (presence of discontinuities like joints, faults, bedding planes, foliations, shear zones and their roughness, nature of infilling material, discontinuity spacing, orthogonal blocks formed by discontinuities etc.), ground water conditions, climate and rainfall history [31, 32, 33, 34].

Various techniques including kinematic analysis, limit equilibrium analysis, rock mass classification system (SSPC system) and probabilistic analysis among others are available for the investigation of rock slope stability [35, 29, 27, 3]. Usually kinematic analysis by utilizing stereographic projection method is carried out before performing detailed study in nearly all slope stability analyses [22, 2]. Kinematic analysis is a geometric method, which employs angular interactions between discontinuity planes to determine the possibility and failure types in a jointed rock mass [21, 38, 19, 16]. In general a rock slope may fail in one of the four failure modes including plane, wedge, circular and toppling. The details pertaining to these failure modes can be found in following references [25, 14, 24, 15, 26, 13].

Stability analysis of rock slopes is a well-researched subject. Studies in past have been performed using a variety of slope stability investigation techniques. Aydan and Kawamoto [4], Bobet [7], Yoon et al. [38], Kentli and Topal [18], Yang and Zou [37], Kulatilake et al. [22], Iqbal et al. [16] and Abu Bakar et al. [1] among others adopted the conventional kinematic and limiting equilibrium approaches for rock slope failure analysis. Park et al. [29] established a probabilistic examination process to analyze the rock slope stability for Interstate Highway 40 (I-40), North Carolina, United States of America. Kim et al. [20] applied the Geographical Information System (GIS) approach for determining the stability of rock cuts, while Pariseau et al. [28], Chen et al. [8] and Li et al. [23] among others adopted the numerical analysis techniques.

The current study was aimed at analyzing the geological discontinuities data collected from three selected rock slopes situated along the Choa Saiden Shah – Kallar Kahar road section. The discontinuities data was then utilized to evaluate the stability of selected roadside slopes through kinematic analysis by employing DIPS (5.1) software.

1.1 Location and Geology of Study Area

The present research work included three different portions of roadside rock slope sites situated along the Choa Saiden Shah-Kallar Kahar road section for slope stability analysis. The locations of slopes are presented in Figure 1.

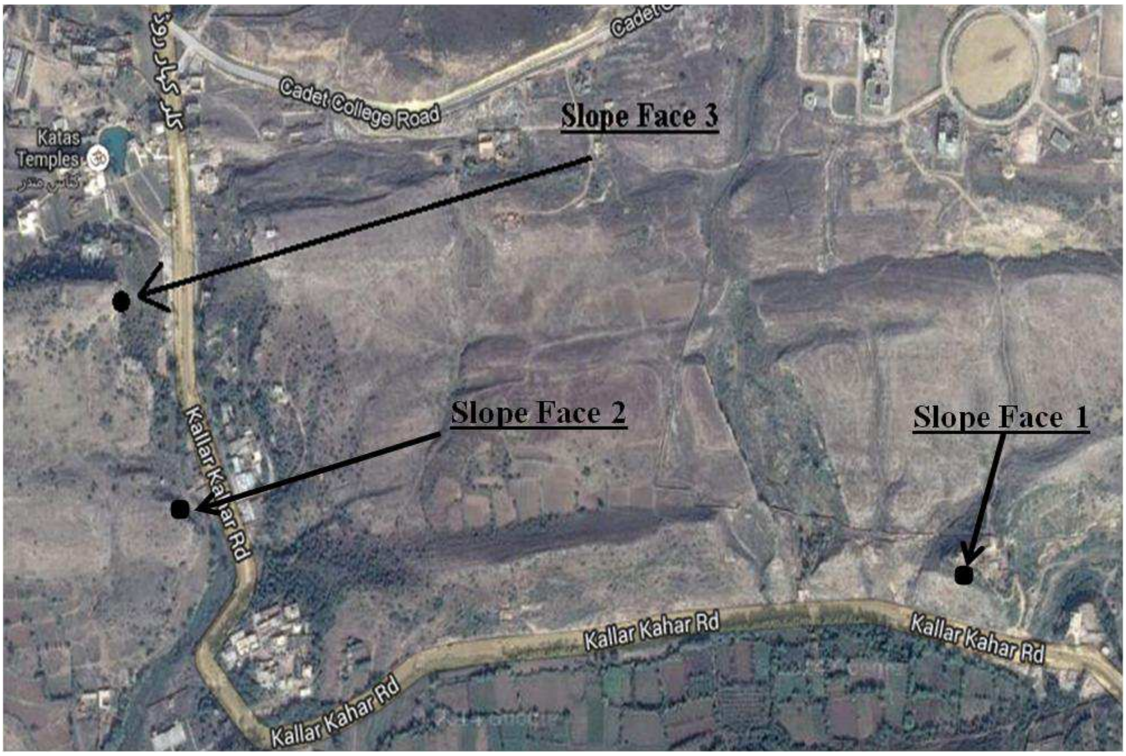


Figure 1. Map showing the location of slope sites 1, 2 and 3

The study area was situated in the eastern part of the Salt Range in which the Eocene succession of the Cenozoic sedimentary sequence of the upper Indus Basin consists of the Nammal Formation, the Sakesar Limestone, and the Chor Gali Formation [6]. The Nammal Formation [11, 12, 10] consists of shale, marl and argillaceous limestone. With increase in limestone beds the Nammal Formation transitionally passes into the overlying Sakesar Limestone [11, 12, 10] which is a grey, nodular to massive limestone having chert in the upper part. It has a thickness ranging from (70 to 300) meters. The Chor Gali Formatoin [30, 10] which lies conformably on the Sakesar Limestone consists of thin bedded grey, partly dolomitized and argillaceous limestone with bituminous odor in the lower part. The upper part of it is composed of greenish, soft calcareous shale with inter beds of limestone. Its thickness ranges from (30 to 140) meters [6].

2. Materials and Methods

For the field mapping of geological discontinuities (joints, bedding planes) the discontinuity orientation data was gathered from the three selected slope sites with the help of a conventional pocket transit Brunton Compass. In this work a combined kinematic analysis approach as mentioned by Wyllie and Mah [36] was used to analyze the discontinuity orientation data for the three rock slopes. The measured values of orientation data (dip and dip directions of these discontinuities and slope faces) for each slope were then analyzed for stability by using DIPS (5.1) software package [9]. The dip and dip directions of slope faces were utilized to draw the daylight envelopes. A friction angle of 37° was evaluated for this stability analysis as per recommendations of Barton [5] and Jaegar and Cook [17]. Figure 2, presents the graphs of pole, contour and principal discontinuity sets pertaining to slope face 1.

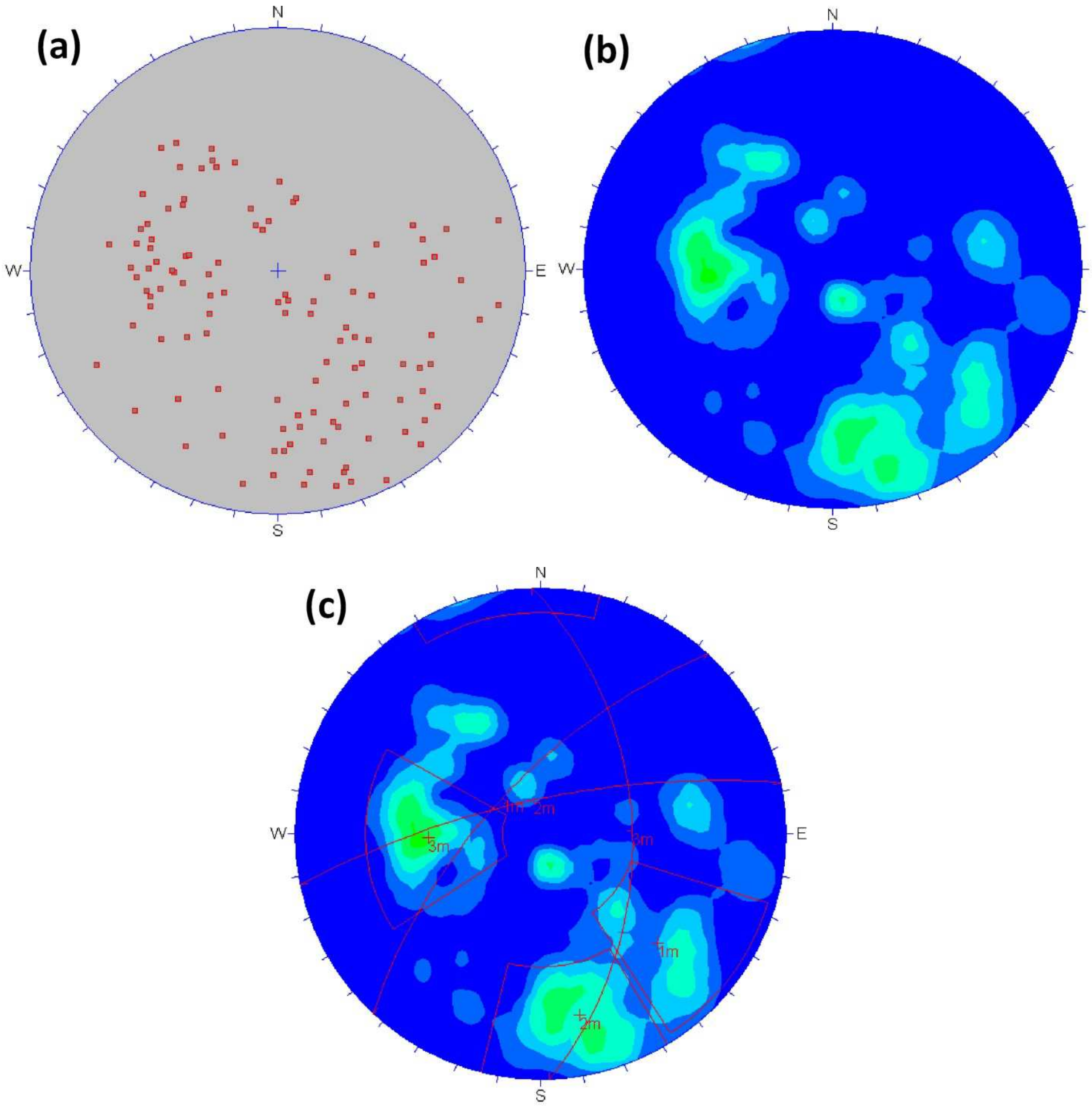


Figure 2. Equal area stereonet projections showing plots of (a) Poles (b) Contours and (c) Sets of Principal discontinuities for slope face 1

3. Results and Discussions

A total of 115, 126 and 159 discontinuity orientations were measured through scan line survey carried out on slope faces 1, 2 and 3 respectively. Table 1 illustrates the principal discontinuity sets and cut face orientation for each slope face. The kinematic analyses of slope face 1, 2 and 3 are presented in Figures 3, 4 and 5 respectively.

Table 1: Orientation Data of Slope Faces

Slope Face Number	Number of Joints Measured	Slope Face Orientation (Dip/Dip direction)	Joint Sets	Principal Joint Sets (Dip/Dip direction)
1	115	(72/162)	1m	(66/313)
			2m	(74/348)
			3m	(49/088)
2	126	(69/135)	1m	(57/329)
			2m	(24/142)
			3m	(73/172)
3	159	(77/141)	1m	(62/330)
			2m	(74/194)

Slope Face 1:

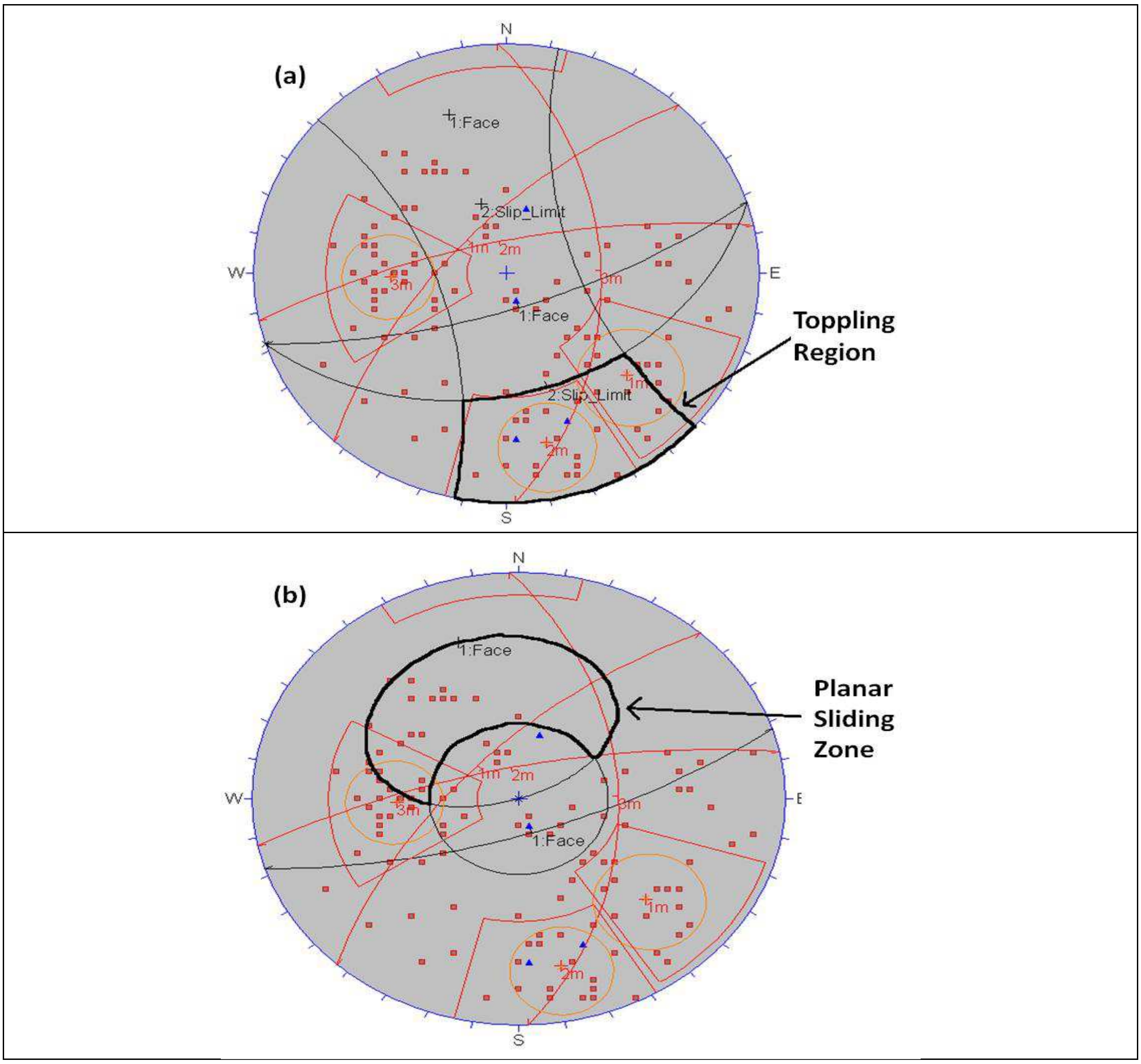
When slope face 1 was assessed for toppling failure (Figure 3a) a portion of joint set 1m and entire joint set 2m fell within the highlighted toppling region bounded by the slip limit and Kinematic bounds. It indicated the slope instability risk of 68 % through toppling failure. Similarly when slope face 1 was analyzed for plane failure (Figure 3b) it was evident that joint sets 1m, 2m and 3m were located outside the daylight envelope for planar failure. Therefore it can be concluded that there exists no risk of planar sliding. Figure 3(c), demonstrates the application of Markland's test for wedge failure. As the intersections of great circles of principal discontinuity planes 1m, 2m and 3m do not fall within the outlined wedge sliding zone therefore slope face 1 is not critical for instability through wedge failure.

Slope Face 2:

Figure (4a) illustrates cut face 2 for toppling failure type. It can be noted that pole of joint set 1m fall at the boundary of outlined toppling region enclosed by the slip limit and Kinematic bounds, therefore this specifies a toppling failure risk of about 50 %. Further it is obvious that poles of joint sets 1m, 2m and 3m are located outside the daylight envelope for plane failure (Figure 4b), therefore it is deduced that slope face 2 is not significant for planar instability. Likewise it is noteworthy in Figure (4c) that the points of intersection of great circles among major discontinuity planes 1m, 2m and 3m do not fall within the outlined wedge sliding zone therefore slope face 1 is also not critical for instability through wedge failure.

Slope Face 3:

In the case of slope face 3 the pole of joint set 1m lies within the highlighted toppling area (Figure 5a) which confirms a significantly high toppling failure threat of 81%. It is noteworthy that poles 1m and 2m (Figure 5b) fall outside the planar sliding zone and the point of intersection of great circles 1m and 2m (Figure 5c) also lie outside the outlined region for wedge sliding zone, therefore the possibilities of plane and wedge failures for cut slope 3 do not exist at all.


(a)
N
W
E
S
1:Face
2:Slip_Limit
1m
2m
3m
Toppling
Region
(b)
Planar
Sliding
Zone

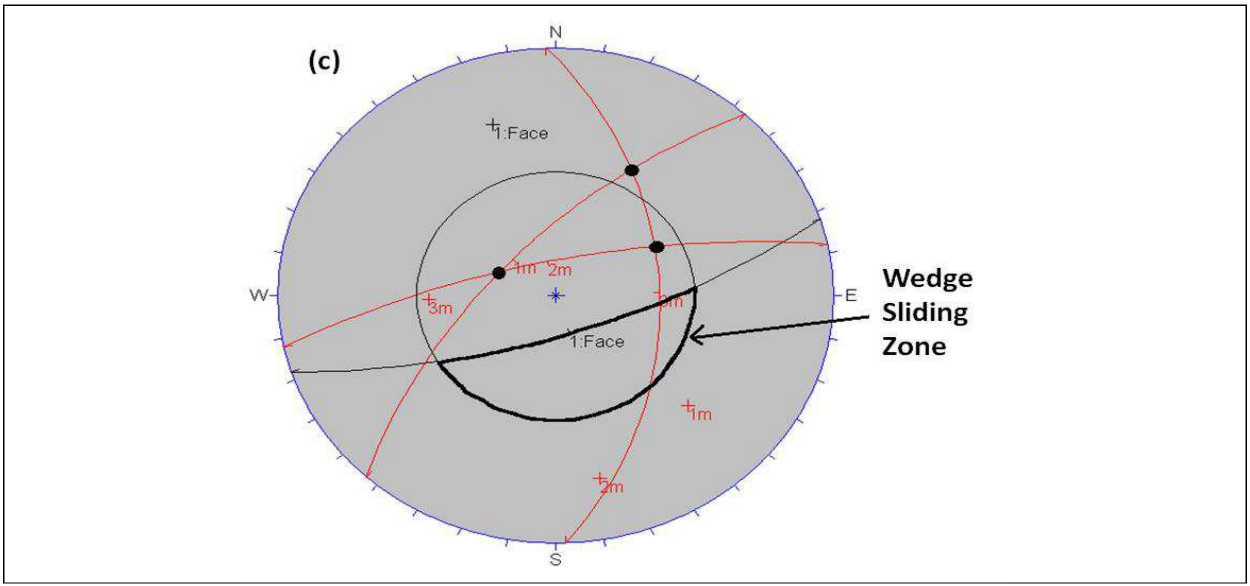
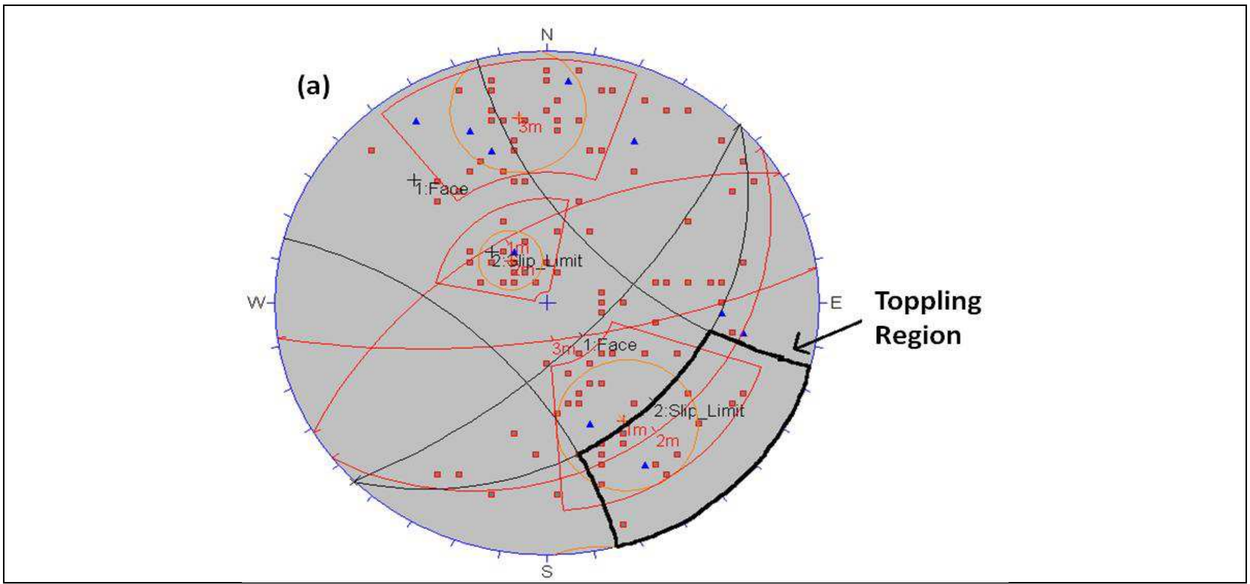


Figure 3. Kinematic analysis of slope face 1



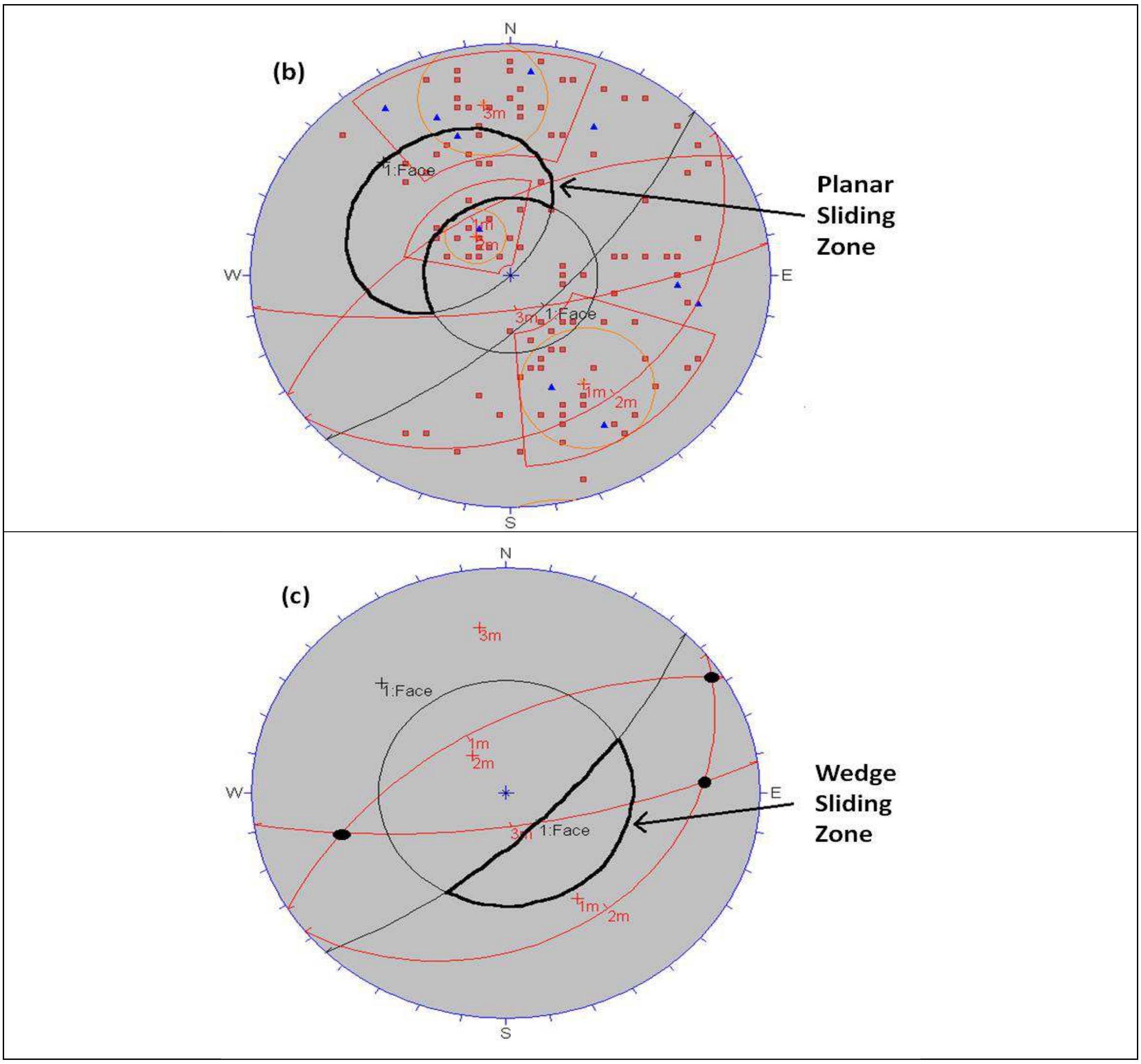
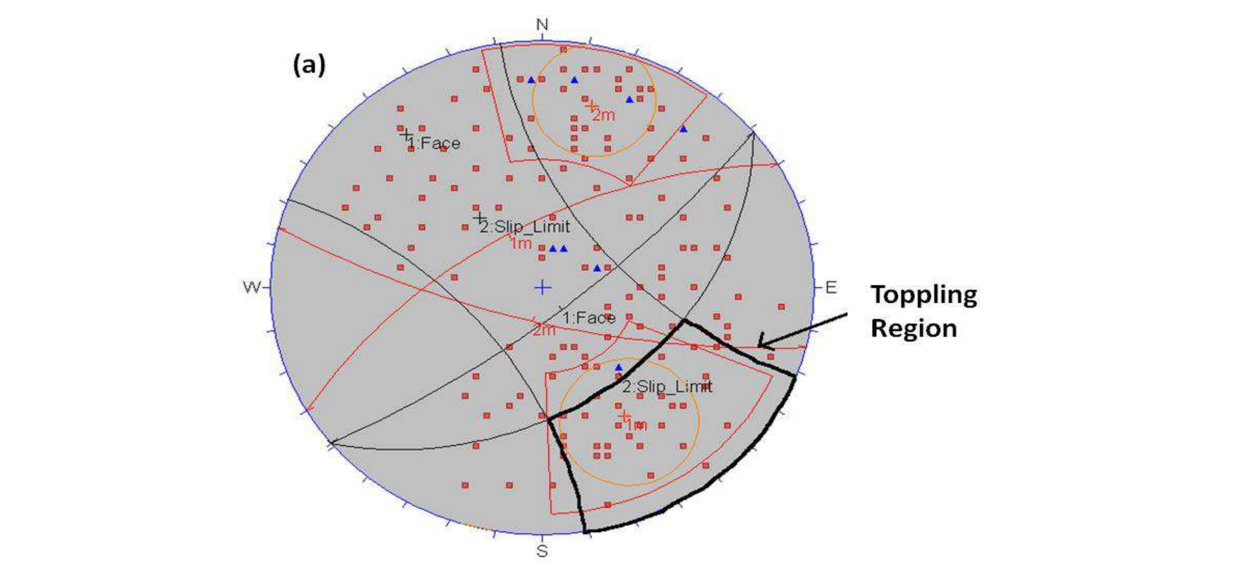


Figure 4. Kinematic analysis of slope face 2


(a)
N
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1:Face
2:Slip_Limit
1m
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Toppling
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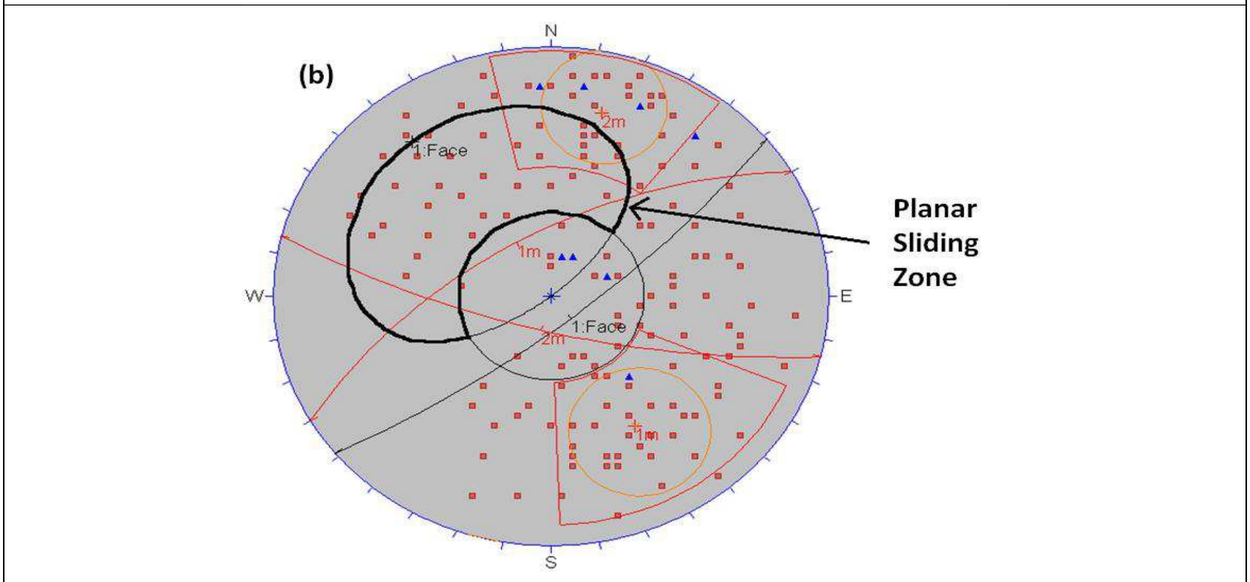
(b)
N
S
W
E
1:Face
1m
2m
Planar
Sliding
Zone

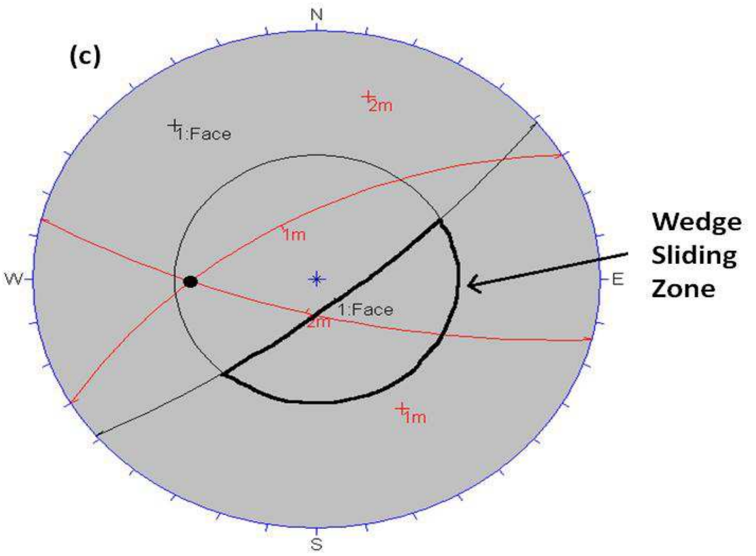

Figure 5. Kinematic analysis of slope face 3

4. Conclusions

Table 2 summarizes the results of kinematic analyses executed on the joints orientation data gathered from three selected rock slope faces situated along the Choa Saidan Shah- Kallar Kahar road section.

Table 2: Summary of kinematic analysis results

Slope Face Number	Stable/Unstable	Failure Mode
1	Unstable	Toppling
2	Unstable	Toppling
3	Unstable	Toppling

The kinematic assessment show that all the cut faces are stable for planar and wedge failures, but are prone to toppling failure risk primarily due to considerable columnar jointing present in the Chor Gali Formatoin and the Sakesar Limestone. Although, there are no reported accidents due to massive slope failures in the study area and its surroundings, but generally minor toppling failures do occur frequently especially in the rainy season (monsoon). The rock slopes under consideration and in the vicinity of the study area are located along a busy road segment posing a constant threat to the passing vehicular traffic therefore, it was deemed necessary to scientifically evaluate the stability of these slopes in order to mitigate the anticipated roadside accidents and their associated damages. Following are some recommendations regarding their stabilization:

- Steel anchors and rock bolts should be introduced to avoid potential toppling failure.
- Surface run off water should not enter into the joints and cracks from the crest of the slope face. Therefore drains and ditches should be employed to ensure adequate drainage of water.
- Rock sheds or rock catch fences should be built over road sides to shield the slope sites under consideration from rock falls and rock avalanches.

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