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Stability Analyses of Exposed Rocks with Respect to Nearby Structures in Akure, Ondo State, Nigeria

Oniyide G. O.
Department of Mining Engineering, Federal University of Technology, Akure, Ondo State, Nigeria
Corresponding Author: Oniyide G. O.

Abstract: *This paper presents the stability analysis of slopes based on translational mechanism of failure. It is aimed at determining the stability of exposed rocks with respect to nearby structures, with case studies on some outcrops in Akure, Ondo State, Nigeria. In this research, oral interviews were conducted to determine if those residing in the areas of study are aware of the dangers associated with rock slope instability. Field investigation was also carried out in order to determine the in situ state of the outcrops under investigation in the three locations, the discontinuities were mapped using window and scanline mapping. The in situ compressive strength was determined, indirectly, using the Schmidt hammer. The results of the analysis indicated that in all the three locations, the criticality of the discontinuities for plane failure is 0% which means that plane slope failure will not occur. In the first location, the criticality of the discontinuities for direct toppling failure is 11.56%, oblique toppling is 20.72% while flexural toppling is 7.69%. For the second Location, the criticality of the discontinuities for direct toppling is 18.71%, oblique toppling is 42.37% while flexural toppling is 31.43%. The joint set one also indicated that its criticality for flexural toppling is 38.45%. Considering the third location, the criticality of the discontinuities for direct toppling is 18.98%, oblique toppling is 50.49% while flexural toppling is 38.75% and the first joint set gave an indication of 100% for flexural toppling failure. It was concluded that for all the locations, location three has the highest critical discontinuities for toppling. If induced further, by some factors contributing to rock slope failure, the outcrop investigated may experience toppling failure, which will lead to loss of lives and property.*

Keywords: *Rock, slope stability, discontinuity, failure.*

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I. Introduction

Rock slope stability analysis has become one of the most important factors considered during the design of new mines and most especially surface mines. This analysis is routinely performed majorly to help in the assessment of safe and functional design of excavated slopes in open pit mining and to determine the equilibrium conditions of natural slopes (Kainthola *et al*., 2013). The major factors affecting the stability of rock slope includes; critical geological features on the rock, the shear strength parameter of critical discontinuities, strength indices of critical discontinuities and pore pressure acting on critical discontinuities (Pentz, 1981; Pattern and Deere, 1971).

A site investigation study should precede any stability study which has to include elements of geological and discontinuity mapping which will help to provide the necessary input data to be used in order to carry out the stability analysis. The collection of data ideally involves rock mass characterization and the sampling of the rock materials for strength and constitutive behavior determination analysis in the laboratory, field observations and in situ measurement. In order to properly conduct a rock stability investigation, and to analyze and evaluate the potential hazard to structures on those exposed rocks or outcrops, it is essential to understand the process and mechanisms driving the instability. Instability mechanism is also known or can also be referred to as Landslide movement which is also called Mass movement. Landslide movements may be in form of falls, topples, slides, spreads or flow (Cruden and Varnes, 1996), and in some cases, it involves different combination of several failure modes. Landslide usually occur in mountainous regions, therefore, structures built close to outcrop may be exposed to danger due to landslide, which may actually result to the collapse of such buildings, leading to a possible loss of lives and properties. Adnan et al (2018) explained that in the 20th century, disasters caused by massive rock slope failures killed more than 50,000 people on a global basis.

The scale of slope stability problem is divided into two types, which are; Gross stability problem: it is classified based on the amount or volume of rock involved .It refers to large volumes of materials which come down the slopes due to large rotational type of shear failure and it involves deeply weathered rock and soil (Brahma, 2009). The second type is the Local Stability problem, which refers to much smaller volumes of material and this type of failure affects one or two benches at a time, in a mine or quarry, due to shear plane jointing slope erosion resulting from surface drainage (Brahma, 2009).

Some of the factors affecting rock slope stability include: Slope Geometry, Geological structures (discontinuities), Lithology, Ground water, Cohesion, Angle of Internal friction, External impulsive forces such as earthquakes, waves, and volcanic eruptions. Vegetation may influence stability through mechanical cohesion and removal of water via evapotranspiration (Abdellah *et al*., 2018). Rock slope stability analyses are performed routinely and directed basically towards assessing the safe and functional design of excavated slopes (for example in open pit mining, road cuts etc) and/or the equilibrium conditions of natural slopes (Kainthola, 2013). According to Abdellah *et al*. (2018), the primary objective of every Rock slope stability analysis is to determine the rock slope conditions and the potential failure mechanisms, the slope sensitivity and susceptibility to different triggering mechanisms, which will also involve comparing different support and stabilization option and to critically address the issue of safety (Goodman and Bray, 1976).

Mariappan *et al*. (2009) suggested that engineers are required to identify the possible factors causing landslides and its impact on society and economy in order to minimize losses. In terms of the risk of life and economic losses as a consequence of slope failure, the major factor to be considered in any slope study and mitigation measures is the proximity of the slope or earth retaining structure to populated areas, traffic and building. Sadagah (2013) stated that slope failures can be prevented in a number of ways, such as using an anchor bolt to hold the rock mass in place or by building a retaining structure. The principle of this method is to use a retaining structure to resist the downward forces of the soil mass, ground anchors or other tie back system may be used together with the retaining structures if the driving forces are too large to resist. One of the slope failure factors is saturation and pore water pressure building up in the subsoil. If drainage system had been provided, the chances of building up pore water pressure and saturation of subsoil can be minimized (Mariappan *et al*. 2010).

Some people may think that once a slope has been standing for years without failure that it will continue to remain intact. This is not true, as it has been proven that natural slopes can fail suddenly without giving any warning even though it has been standing for years without signs of failure but the Factor of Safety may be low and near the threshold. Hence, it is not safe to assume that natural slopes are safe until it has been investigated and analyzed.

In order to study the different types and scales of failure and instability, it is of great importance to know the different types of failure, the factors affecting them in detail and how they tend to affect structures, built on or around such rock outcrops. This study investigates some instabilities which may occur with respect to outcrops and structural buildings built on or around the outcrops and the major slope instability’s drive mechanisms. Three sites in Akure were used as the study area, which are: Location 1 (Olufoam Street), Location 2 (Express road along Oke-Ijebu Street) and Location 3 (Abusoro street Ijoka).

Akure is a city in South-Western Nigeria, and is the largest city and capital of Ondo State. The city has a population of 484,798 as at the 2006 population census. It lies between Latitude 7°15' 9.22" N and Longitude 5°11' 35.23" E (Olajuyigbe, 2015) (Figure 1). Rock engravings dating back to the Mesolithic period have been discovered on the outskirts of Akure. Major rocks in Akure are granite rocks and Charnokite (Ademeso, 2009). The granite rocks which are member of the older granite suit occupy about 65% of the total area of Akure. Three varieties are recognized, the fine grained biotite granite, medium to coarse grained, non-porphyritic biotite-hornblende granite and coarse- porphyritic biotite -hornblende granite.

II. Materials And Method

Materials used for data field acquisition are; L-type Schmidt hammer (as recommended by ISRM 1981), Global Positioning System (GPS), compass clinometer, measuring tape, cutlasses and a field-book. Rocscience software (Dip 7.0) was used for the Result analysis.

1.1 METHOD

The research methodology involves discontinuity mapping, other physical measurements and observations in the study areas. The data from the field were then analyzed.

1.1.1 Field Work Procedure

At each Location, after sighting the outcrop that is near to structural buildings that is to be investigated, cutlasses were used to create a path-way in order to gain access to the outcrop. After climbing the outcrop, the Global Positioning System was used to obtain the coordinates and the elevation. A section of the outcrop was mapped using scanline mapping technique, compass clinometer was used to obtain the dip and dip direction of each point of discontinuity, and all the data obtained were recorded in the field book. The Schmidt hammer was also used to obtain the rebound values which is used to determine the in situ compressive strength. A total number of fifty (50) rebound values were obtained from each location.

For the discontinuity mapping, an expanse front section of the rock outcrop facing the nearby structures was used, the meter rule was placed at the average trunk height of an average male height (1.5 m) from one end of the rock face to the other. All discontinuities that falls within this line was mapped, and the rebound hardness values of the outcrop were also obtained using the Schmidt hammer. Distance between the outcrop and the nearest building was obtained using the measuring tape. All data obtained were recorded in the field book. A total number of hundred (100) discontinuities were investigated for the discontinuity mapping at each location. The distance of buildings from the base of the outcrops at locations 1, 2 and 3 are 9.5 m, 2.63 m and 2.81m respectively.

1.1.2 Data Analysis

The Rocscience software, Dip 7.0 which was used for data analyses, has an interface where all the field investigation data are to be imputed, such data could be in dip and dip direction, plunge and trend and so on. The Project settings was the first window to be accessed, in which the data type was set to dip and dip direction, also the field data that has been typed in Microsoft Excel format was copied into the data interface and other field data columns were also added which includes; type of discontinuity, joint spacing, aperture and the persistence. After the imputation of all the data, the vector plot, the contour plot (Both 2D and 3D), the Rosette plot interface were accessed and the results were exported in an image format (JPEG). Also the kinematic analysis for the slope monitoring investigation were done using the Kinematic analysis interface in which the contour 2D plot and the vector plot was used to obtain some joint sets which actually aided the analysis. The data were analyzed for planar failure and Toppling (Direct and Flexural) failure. Circular failure cannot occur because the outcrops investigated are Granite rocks, and circular failure usually occurs in soils (unconsolidated rock materials). Therefore, the analysis for circular failure was not considered. All results obtained from the Kinematic analysis were exported in image format (JPEG) in order to avoid incompatibility of the results file.

III. Results And Discussion

The kinematic analysis for plane and toppling failure was based on the following basic facts; that the Slope angle of all the outcrops under investigation is 70° and the frictional angle is 30°, while taking the percentage critical point at which failure will occur to be 50%, and the lateral limit of all the set of discontinuities is 20°. It should be noted that this analysis focused on the deterministic aspect of Rock slope instability, and does not include the probabilistic aspect.

For the first location, the maximum density of all the discontinuities is 9.52% while the counting circle size is 1.0%. The kinematic analysis for planar failure showed that the point of intersection of the two joint sets did not fall in the plane of failure, therefore plane failure will not occur. While the kinematic analysis for Direct (Block) toppling based on the intersection showed that the total points of intersection is 2993 while the critical points between the joint sets is 346, which showed that the percentage of direct toppling occurrence at the time when this data was obtained is 11.56%. The analysis also showed that the total set of discontinuities for oblique toppling based on their intersection is 2993 showing that the total number of critical set intersection is 620 which actually mean that the percentage of oblique toppling occurrence is 20.72%. The kinematic analysis for flexural toppling also showed that out of 78 total set of discontinuities, only 6 is termed as critical sets, which means that the percentage of flexural toppling failure occurrence is 7.69%. Through this analysis it can be inferred that, at this present state of the outcrop, the rock outcrop will not experience any form of instability, either plane or toppling failure at present, but if the criticality of the joint sets is induced by any of the factors that can contribute to rock instability, surely toppling failure will occur, and such failure may actually result to the loss of lives and properties of those residing very close to the rock outcrop (Figures 2 to 4).

For the second location, The Maximum density of all the discontinuities is 16.23% while the counting circle size is 1.0%. The Kinematic analysis for plane failure also showed that the point of intersection of the joint sets did not fall in the plane of failure, therefore showing no sign of plane failure occurrence, while the Kinematic analysis for Direct (Block) toppling based on the intersection of discontinuities showed that the total points of intersection is 2410 while the critical points between the joint sets is 451, which actually translates that the percentage of direct toppling occurrence based on the intersection of the discontinuities is 18.71%. The analysis also showed that the total set of discontinuities for oblique toppling based on their intersection is 2410, and that the total number of critical sets based on the intersection is 1021 which actually mean that the percentage of oblique toppling occurrence is 42.37%, falling a little bit short of the percentage critical point at which failure will occur. The kinematic analysis for flexural toppling also showed that out of a total of 70 set of discontinuities, 22 is termed as critical, which means that the percentage of flexural toppling failure occurrence is 31.43%, while the first joint set which comprises of 39 set of discontinuities, contains 15 critical set discontinuities, which showed that the percentage of flexural toppling failure occurrence is 38.45%. The toppling failure analysis of the rock outcrop in the second location actually define a unique kind of toppling failure mechanism, in which through this analysis, it can be inferred that as the block of rocks flexures, all other blocks embedded in the same block are also conforming to the same mode of failure. This scenario can actually induce the toppling failure rapidly and through this analysis it can be inferred that at this present state of the outcrop, it will not experience any form of instability, either plane or toppling failure, but if the criticality of the joint sets are induced by any of the factors that can contribute to direct toppling failure, surely toppling failure will occur, which will actually results to the loss of lives and properties of those residing very close to the rock outcrop since the level of criticality in on a rise (Figures 5 to 7).

For the third location, The Maximum density of all the discontinuities is 12.70% while the counting circle size is 1.0%. The Kinematic analysis for plan failure also showed that the point of intersection of the joint sets did not fall in the plane of failure, therefore showing no sign of plane failure occurrence while the Kinematic analysis for Direct (Block) toppling based on the intersection of discontinuities showed that the total points of intersection is 3145 and the critical points between the joint sets is 597, which implies that the percentage of direct toppling occurrence based on the intersection of the discontinuities is 18.98%. The analysis also showed further that the total set of discontinuities for oblique toppling based on their intersection is 3145, and that the total number of critical sets based on the intersection is 1588 validating that the percentage of oblique toppling occurrence is 50.49%, hereby exceeding the percentage critical point at which failure will occur. This implies that the outcrop that was investigated in the third location is already on the brink of experiencing toppling failure, if any rock slope instability triggering factor should affect the outcrop, even at a minimum level, toppling failure will occur, hereby putting the lives of those residing very close to the outcrop in danger. The kinematic analysis for flexural toppling also showed that out of a total of 80 set of discontinuities, 31 is termed as critical, which means that the percentage of flexural toppling failure occurrence is 38.75%, while the first joint set which comprises of 19 set of discontinuities, contains 19 critical set discontinuities, which placed the percentage of flexural toppling failure occurrence to be on the 100% mark. The toppling failure analysis of the rock outcrop in the third location is an indication of the outcrop’s critical state and the danger in which those living very close to this outcrop are exposed to. Therefore, through this inference, the rock outcrop is on the brink of experiencing toppling failure, although the physical state of the outcrop may still prove otherwise, but in most situations, rock slope instability may occur without giving any physical indications, but the true state of the failure is not classified based on the physical state. In Slope stability monitoring exercise, this notion is widely acceptable for slope management programs in open pit mines and quarries (Figures 8 to 10).

IV. Conclusion

The analysis of the data from discontinuity mapping obtained from the outcrops under investigation in Olu-foam residential area, Express road along Oke-Ijebu Street and Abusoro-Ijoka residential area revealed that on all the outcrops, plane failure will not occur. This is because the analysis did not give any state of criticality on any of the discontinuities while the test for toppling failure clearly showed that all the three locations have the potential of experiencing different forms of toppling failure. Though the level of the criticality is still minimal at this present state but if these outcrops are continually exposed to factors that can induce toppling failure, such outcrop will tend to experience toppling failure. The result analysis revealed that the third location has criticality of the discontinuities on the outcrop and has a higher probability of toppling failure occurrence.

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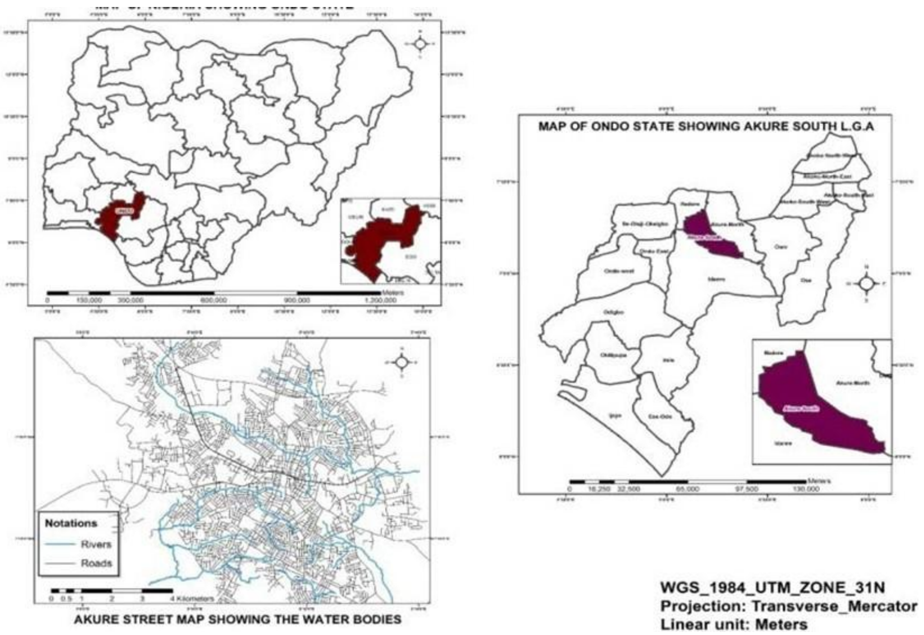
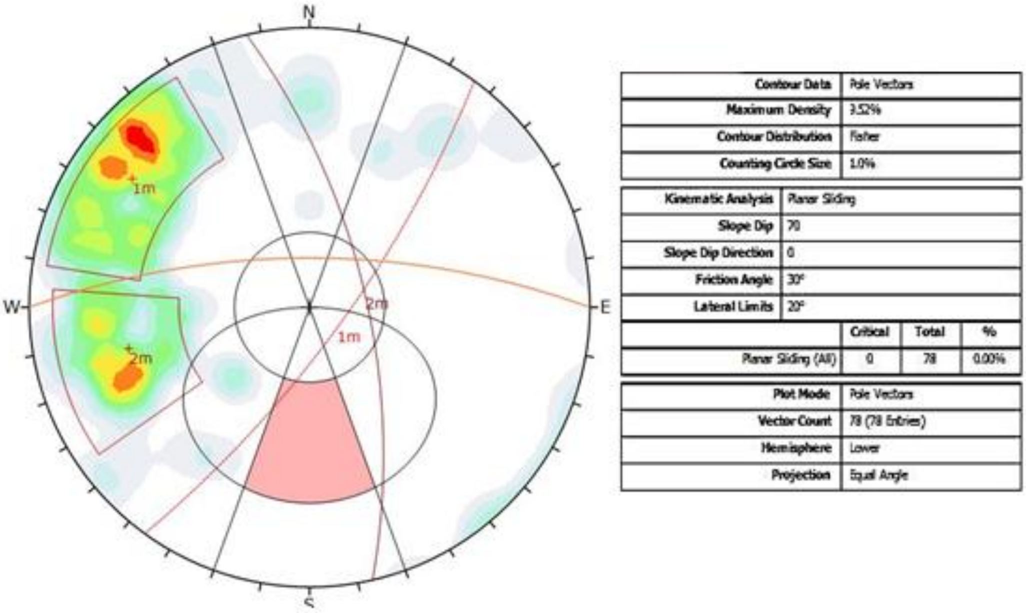


Figure 1. The map of Nigeria, Ondo State and Akure, showing the street map of Akure with the water bodies (Olajuyigbe et al. 2015)



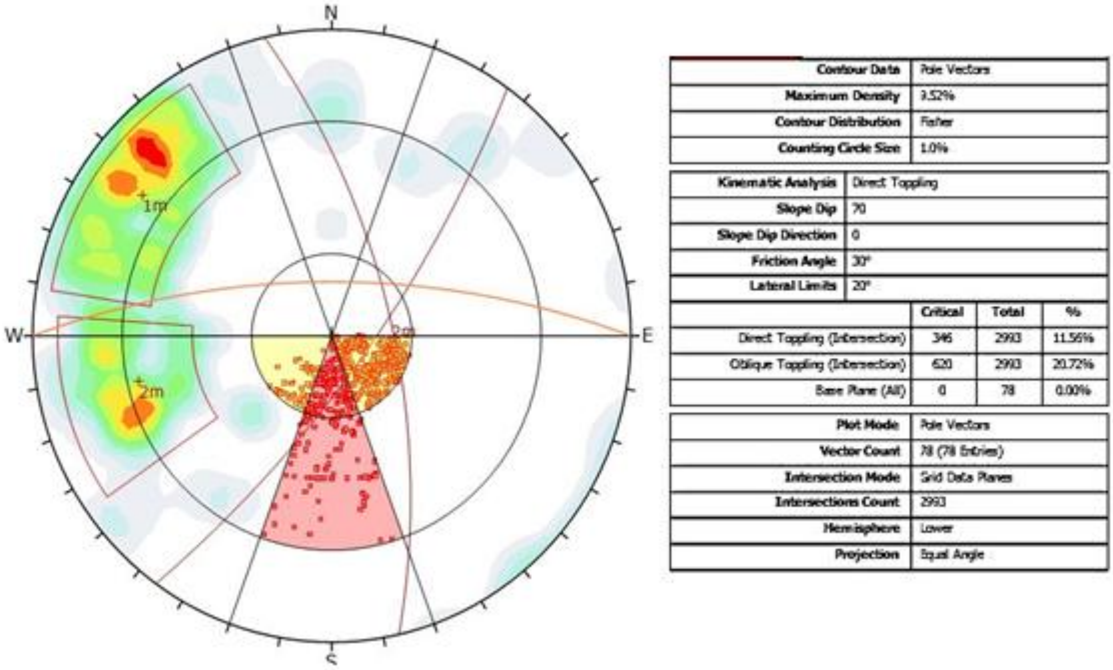
Contour Data	Pole Vectors
Maximum Density	3.52%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Planar Sliding
Slope Dip	70
Slope Dip Direction	0
Friction Angle	30°
Lateral Limits	20°

	Critical	Total	%
Planar Sliding (All)	0	78	0.00%

Plot Mode	Pole Vectors
Vector Count	78 (78 Entries)
Hemisphere	Lower
Projection	Equal Angle

Figure 2. Stereographic plot showing kinematic analysis for planar failure in location 1



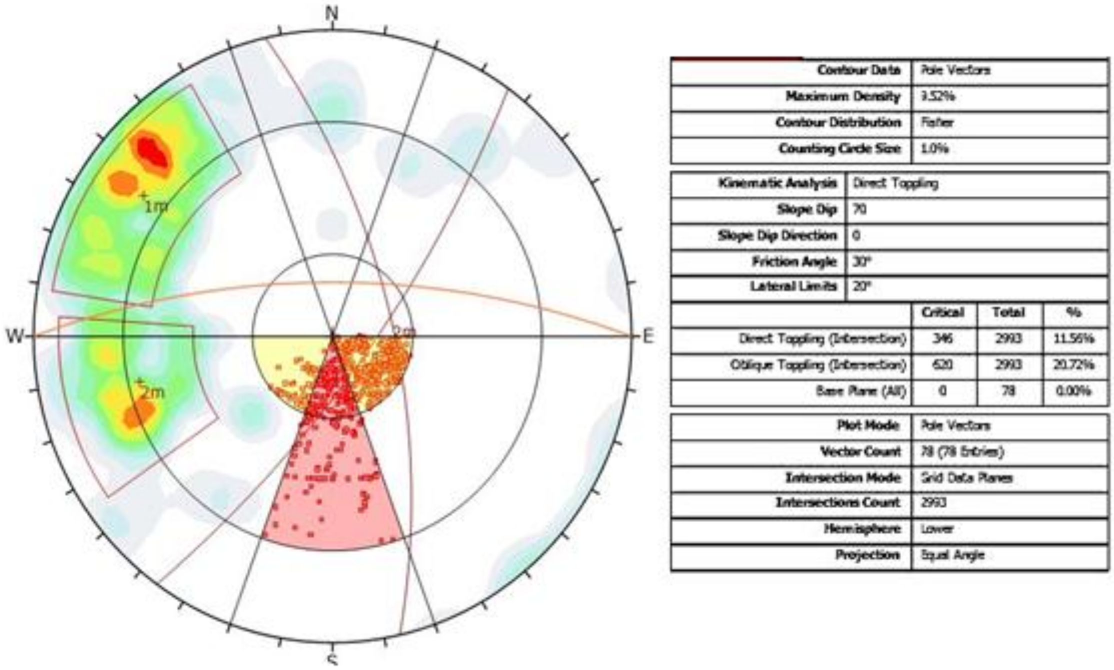
Contour Data	Pole Vectors
Maximum Density	3.52%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Direct Toppling
Slope Dip	70
Slope Dip Direction	0
Friction Angle	30°
Lateral Limits	20°

	Critical	**Total**	**%**
Direct Toppling (Intersection)	346	2993	11.56%
Oblique Toppling (Intersection)	620	2993	20.72%
Base Plane (All)	0	78	0.00%

Plot Mode	Pole Vectors
Vector Count	78 (78 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	2993
Hemisphere	Lower
Projection	Equal Angle

Figure 3. Stereographic plot showing kinematic analysis for flexural toppling failure in location 1



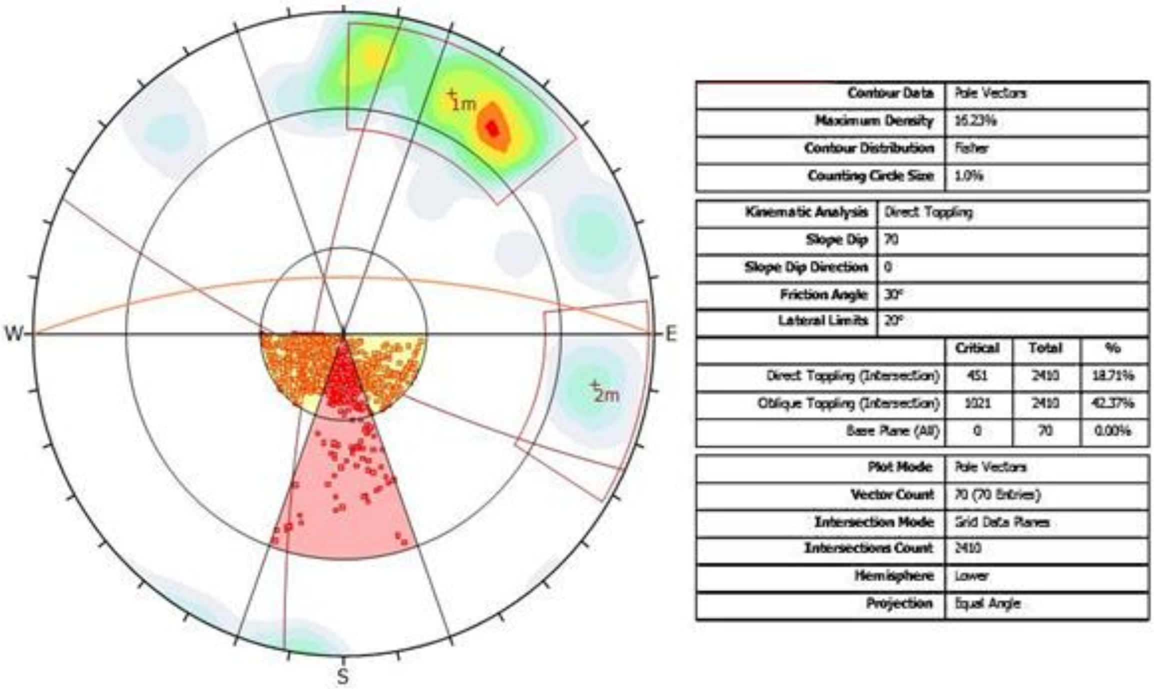
Contour Data	Pole Vectors
Maximum Density	3.52%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Direct Toppling
Slope Dip	70
Slope Dip Direction	0
Friction Angle	30°
Lateral Limits	20°

	Critical	**Total**	**%**
Direct Toppling (Intersection)	346	2993	11.56%
Oblique Toppling (Intersection)	620	2993	20.72%
Base Plane (All)	0	78	0.00%

Plot Mode	Pole Vectors
Vector Count	78 (78 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	2993
Hemisphere	Lower
Projection	Equal Angle

Figure 4. Stereographic plot showing kinematic analysis for direct toppling in location 1



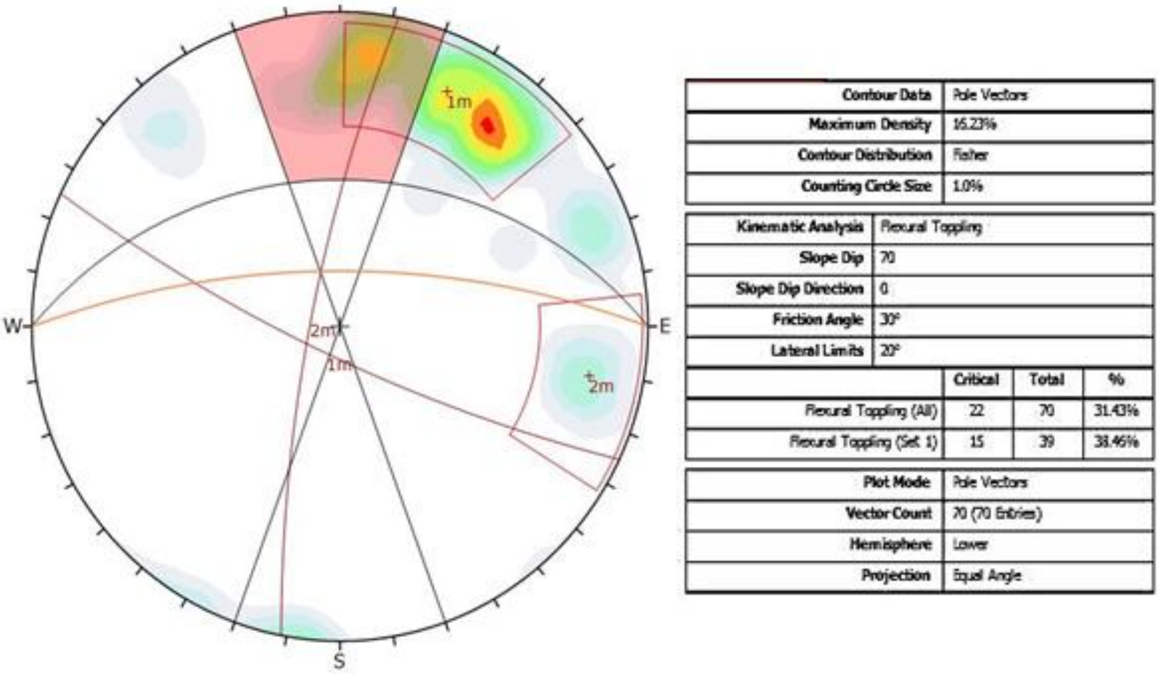
Contour Data	Pole Vectors
Maximum Density	16.23%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Direct Toppling
Slope Dip	70
Slope Dip Direction	0
Friction Angle	30°
Lateral Limits	20°

	Critical	Total	%
Direct Toppling (Intersection)	451	2410	18.71%
Oblique Toppling (Intersection)	1021	2410	42.37%
Base Plane (All)	0	70	0.00%

Plot Mode	Pole Vectors
Vector Count	70 (70 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	2410
Hemisphere	Lower
Projection	Equal Angle

Figure 5. Stereographic plot showing kinematic analyses for direct toppling in location 2



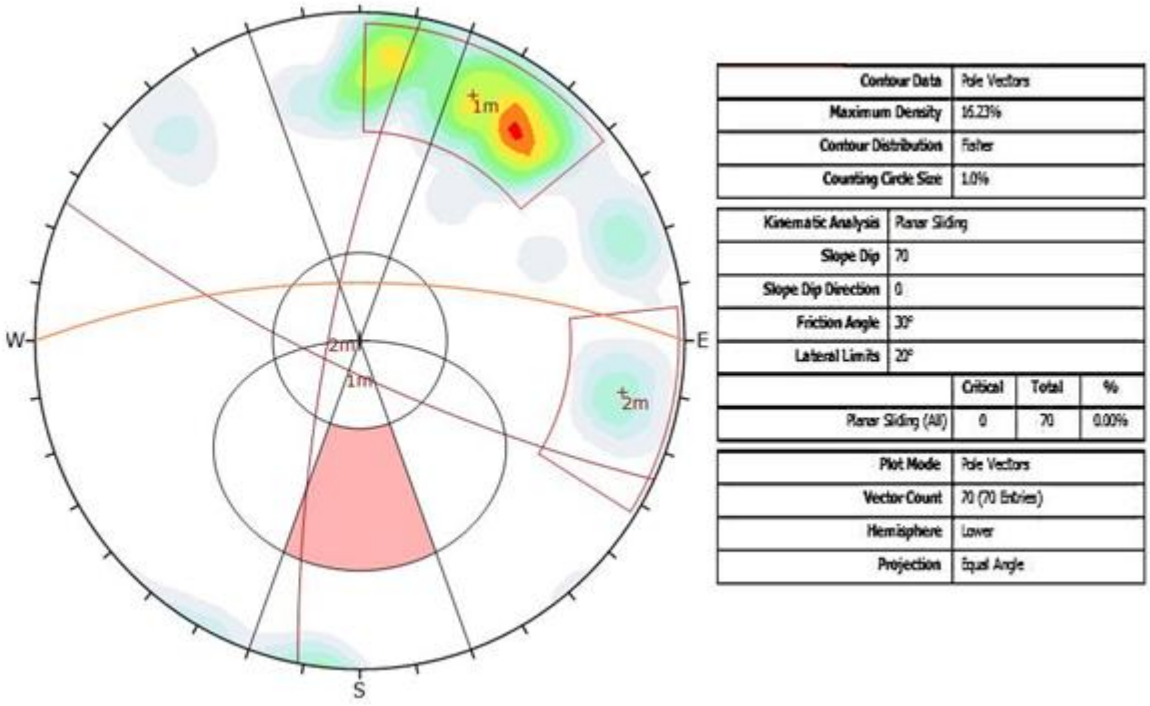
Contour Data	Pole Vectors
Maximum Density	16.23%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Flexural Toppling
Slope Dip	70
Slope Dip Direction	0
Friction Angle	30°
Lateral Limits	20°

	Critical	Total	%
Flexural Toppling (All)	22	70	31.43%
Flexural Toppling (Set 1)	15	39	38.46%

Plot Mode	Pole Vectors
Vector Count	70 (70 Entries)
Hemisphere	Lower
Projection	Equal Angle

Figure 6. Stereographic plot showing kinematic analysis for flexural toppling failure location 2



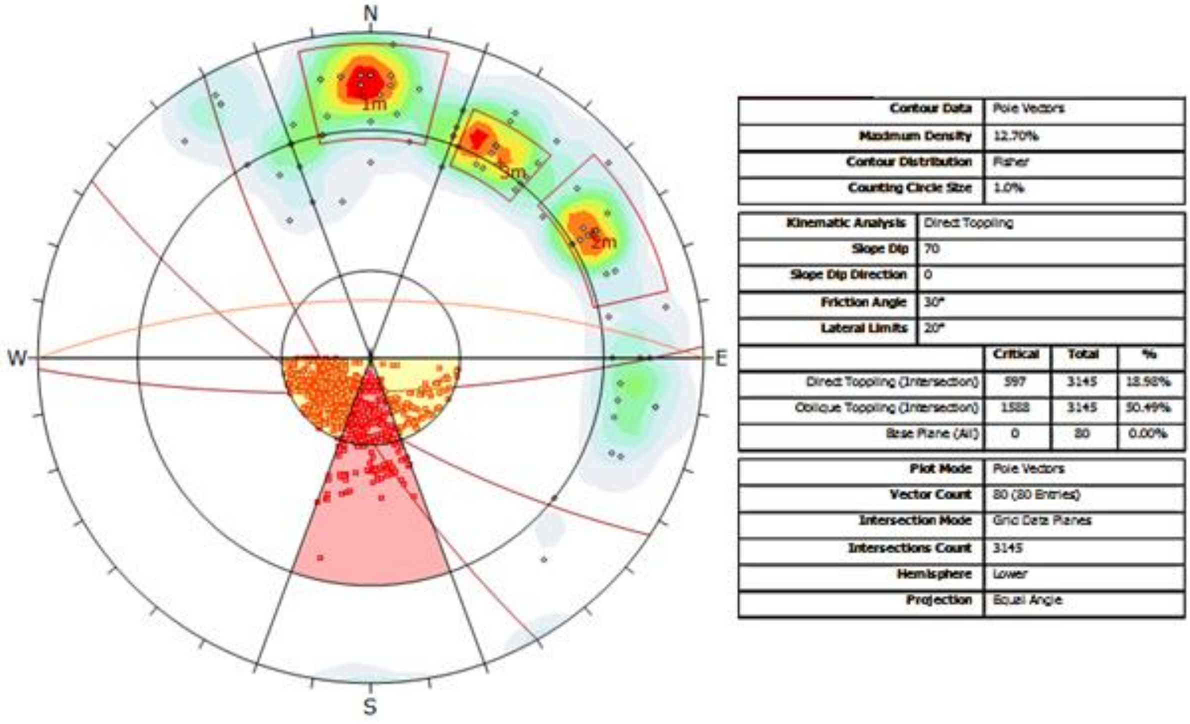
Contour Data	Pole Vectors
Maximum Density	16.23%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Planar Sliding
Slope Dip	70
Slope Dip Direction	0
Friction Angle	30°
Lateral Limits	20°

	Critical	**Total**	**%**
Planar Sliding (All)	0	70	0.00%

Plot Mode	Pole Vectors
Vector Count	70 (70 Entries)
Hemisphere	Lower
Projection	Equal Angle

Figure 7. Stereographic plot showing kinematic analysis for planar failure in location 2



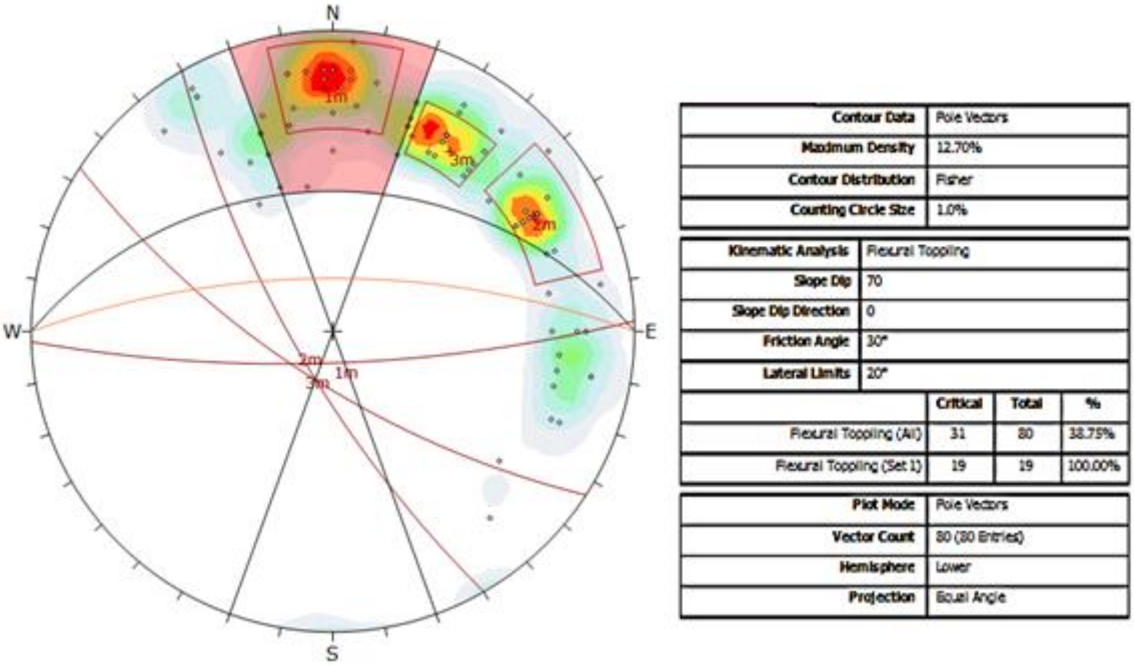
Contour Data	Pole Vectors
Maximum Density	12.70%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Direct Toppling
Slope Dip	70
Slope Dip Direction	0
Friction Angle	30°
Lateral Limits	20°

	Critical	**Total**	**%**
Direct Toppling (Intersection)	597	3145	18.98%
Oblique Toppling (Intersection)	1588	3145	50.49%
Base Plane (All)	0	80	0.00%

Plot Mode	Pole Vectors
Vector Count	80 (80 Entries)
Intersection Mode	Grid Data Planes
Intersections Count	3145
Hemisphere	Lower
Projection	Equal Angle

Figure 8. Stereographic plot showing kinematic analysis for direct toppling in location 3



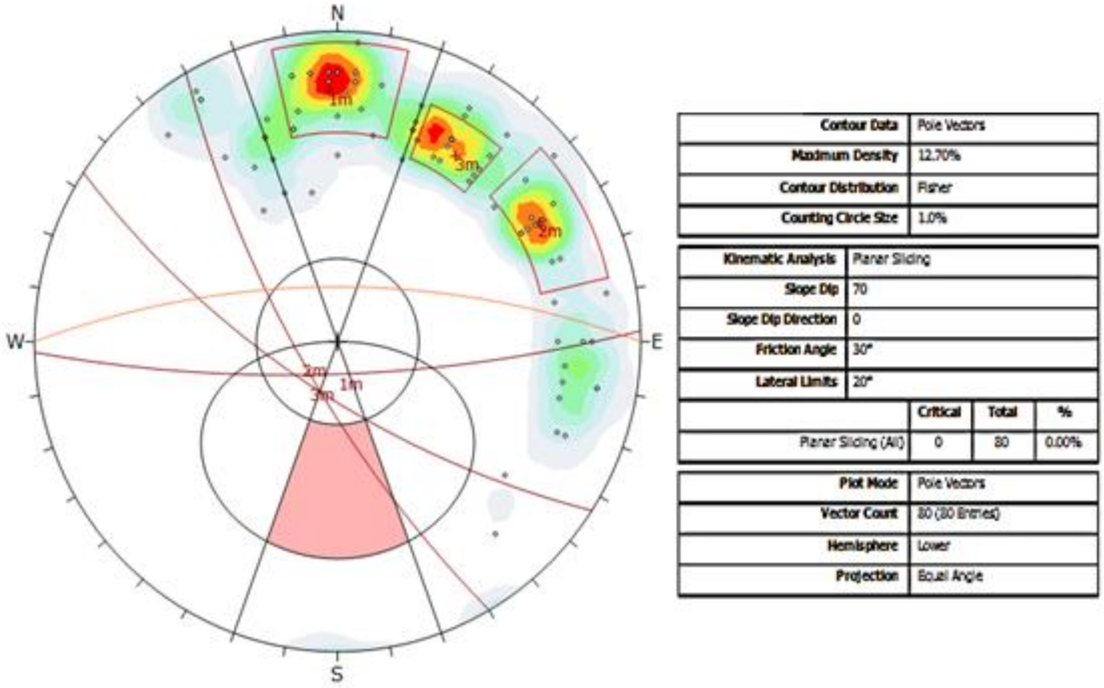
Contour Data	Pole Vectors
Maximum Density	12.70%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Flexural Toppling
Slope Dip	70
Slope Dip Direction	0
Friction Angle	30°
Lateral Limits	20°

	Critical	Total	%
Flexural Toppling (All)	31	80	38.75%
Flexural Toppling (Set 1)	19	19	100.00%

Plot Mode	Pole Vectors
Vector Count	80 (80 Entries)
Hemisphere	Lower
Projection	Equal Angle

Figure 9. Stereographic plot showing kinematic analysis for flexural toppling failure in location 3



Contour Data	Pole Vectors
Maximum Density	12.70%
Contour Distribution	Fisher
Counting Circle Size	1.0%

Kinematic Analysis	Planar Sliding
Slope Dip	70
Slope Dip Direction	0
Friction Angle	30°
Lateral Limits	20°

	Critical	Total	%
Planar Sliding (All)	0	80	0.00%

Plot Mode	Pole Vectors
Vector Count	80 (80 Entries)
Hemisphere	Lower
Projection	Equal Angle

Figure 10. Stereographic plot showing kinematic analyses for planar failure in location 3

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