Failures and Remediation of Rock Slope Stability Problems and Remediation at Earthquakes at Part of Werka Descent Road West of Saudi Arabia

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FAILURES AND REMEDIATION OF ROCK SLOPE STABILITY PROBLEMS AND REMEDIATION AT EARTHQUAKES AT PART OF WERKA DESCENT ROAD WEST OF SAUDI ARABIA

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ABSTRACT

Werka descent road lies at western region of Saudi Arabia. It is constructed two years ago harmed due to rainstorm and absence of remedial measures. Werka mountainous road subjected to failures of man-made rock slope faces, and debris flow along both sides of 8-m width road. Intensive geotechnical study includes the *RMR* and *GSI* rock masses classifications were applied indicates that the rocks are medium to poor quality. The integrated techniques such as graphical method, modeling, and simulation were utilized to assess rock slope failures and rockfalls by using *DIPS*, *RocFall*, *RocPlane* and *Swedge* programs, and recommend the remedial measures for failures. The seismic coefficients of 0.1 to 0.4 were taken into consideration in modeling. Debris flows from higher elevation were a result of the poor rock quality. A manmade slope cut were studied and modeled utilizing the integrated techniques. The analyses indicate that the intensity of rainfall, joints set attitudes with the slope face attitude, J_v, block size, block shape, specific gravity, coefficients of restitution and the slope geometry are the main factors in rock slope failure, rockfall and debris flows problems.

INTRODUCTION

The urbanization and development strategy of the government of the Kingdom of Saudi Arabia is basically grounded on the construction of modern roads and highways network. In addition, a great deal of attention has been given to construct a number of descent roads, where road cuts and bridges as well as tunnels through the higher mountains in these descents. These descent roads play a vital role in connecting the various parts of the Kingdom together, and the ease of transportation. Such routes are connecting to the Red Sea coastal plain at the west with the high-rising mountains at the east.

These descent roads, especially those across areas of various topography of high relief are similar to those in the western part of the country, are currently suffering from common rockfalls and landslides.

Engineering projects often require the excavation of the rock cuts that must be safe for rockfalls and large-scale slope instability, during both construction and operation stages. An example of the difficult descent is under investigation of this research study. Many rock slope failures and rockfalls locate at Tabuk governate roads, which is locally known as Werka descent (Fig. 1) located at Alkharar town.

Fig. 1. Werka descent at Alkharar town, Tabuk governate.

The geographic location of the whole study area at Al-Wajh quadrangle, Tabuk governate is shown in (Figs. 2 and 3). Werka descent 8-meter road is constructed two years ago north of the Kingdom is frequently subjected to rock slopes failures, rockfalls, and flooding, especially during rainy seasons in the recent years. The GPS technology used to trace Werka descent road alignment and plotted on the Google earth image (Fig. 3). The specific location starts at located at 26° 10’ 3.53”N and 37° 24’ 1.77”E and ends at 26° 10’ 45.28N and 37° 23’ 5.96”.

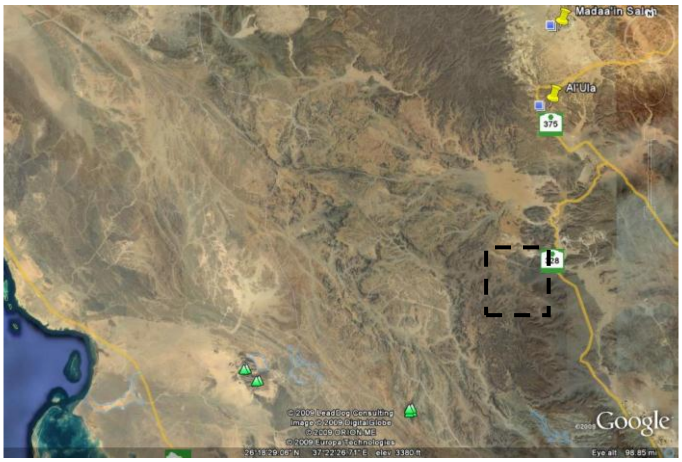

Fig. 2. Satellite image show the location of the whole study area at Al-Wajh quadrangle, the dashed rectangle.

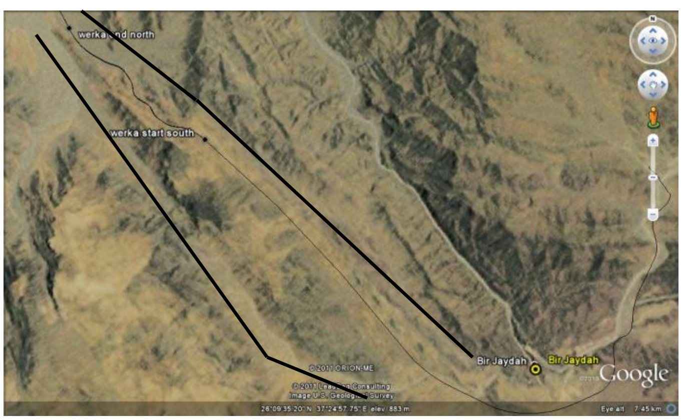

Fig. 3. Location of the Werka descent is shown on Google earth image. The black thin line is the present road in the study area, and the black thick line represents the faults.

BRIEF GEOLOGY

The whole study area in general lies at southeast corner of Al-Wahj quadrangle, in the northwestern Al Hijaz between latitudes 26°00’N and 27°00’N, and longitudes 36°00’E and 37°30’E. The rock masses are underlain by late Precambrian rocks. The Arabian Shield consists of folded, metamorphic plutonic and stratified rocks. The Precambrian lithostratigraphic succession of the Al-Wajh quadrangle is explained by Bryan Davies (1985). The study area of Werka descent lies at Hajr formation, the oldest unit of Bayda group.

<u>Metamorphism</u>

Most of the Precambrian rocks of the quadrangle were regionally metamorphosed to the low and middle greenschist facies. However, amphibolite-grade metamorphism tool place in some complexes, zones and groups. Most of the rocks along Werka descent road are altered to chlorite and clays.

<u>Structural geology</u>

Two phases of major folding have been recognized in the quadrangle. The first-phase folds are low-dipping axial surfaces striking about 90°E. The second-phase folds has variably dipping axial surfaces striking between N 45°S and N 70°W, which affect Werka descent.

In the southeast corner of the quadrangle, major faults with strikes ranging from N 80°W to N 45°W extend into the zone of Najd faults that trend about N 45°W across the Precambrian shield of northern Arabia (Brown & Jackson, 1960; Blank, 1977; and Moore, 1979).

The rocks at Werka descent lie between two faults striking between N 45°S and N 70°W. A number of minor faults (Fig. 3) are located in the area at small and micro scale. Faults strikes are in the same direction of major faults in the quadrangle between N 45°S and N 70°W. Schistosity direction of the rocks has the faults attitudes. Folds strikes are taking the same directions as faults.

ROCK MASSES QUALITY

The rock slope under investigation which is a part of Werka descent road is about 200 m long. The rock masses at THIS station are rigid, altered. The rock masses are metabasalt dry to dump, medium to poor quality after corrections, according to *RMR* classification system (Bieniawski, 1989), and *GSI* = 45 (Hoek 1994; Hoek et al. 1995; and Hoek 2007).

The technical properties of the rocks are as follows: the joints friction angle is 34°, the rock material 2.76 kg/m^3, compressive strength =72 MPa, medium to highly weathered, *RQD* = 82 in general, joint spacing = 5.26 j/m, block size = 0.04 m^3, *RMR* = 34, i.e. poor rock mass quality, *GSI* = 15 to 35.

The rock slope along the road cut at slope angle 85° and 7 m height, suffers from frequent failures on the road, mainly in rainy seasons (October to April). No support measures were taken at the site (Fig. 4).

The graphical method (Hoek and Bray 1981) shows that the factor of safety is critical along man-made slopes. The stereonet made using *DIPS* software (Fig. 5) show the friction angles at dry and wet seasons are 35° and 25°, respectively.

It is obvious that such slope cut suffer from many events of rockfalls originated from upper slope elevations, which are potentially source areas causing problems to the road commuters and vehicles.

ROCK SLOPES MODELING

Location of the stations along the Werka descent is traced by the GPS. It happens that in some places rock slope failures were so heavy at one spot, and it took place on both sides of the road and blocked it (Fig. 4). The results of the stereographic projection show that wedge, plane and toppling failures took place, (Fig. 5). Failures types and directions are given in Table 1.

<u>*Plane failures modeling at north east side*</u>

The great circles of the prevailing joints sets show that the plane failures (Fig. 5) could take place, as they are close to the friction angle value at wet conditions. The great circles also indicate that a number of wedge failures will take place at rainfall conditions.

Fig. 4. Rock slope failures at the northeast (right) and southwest (left) slope face cuts, 2008. Rock blocks were removed after the rainfall aside to clear the road.

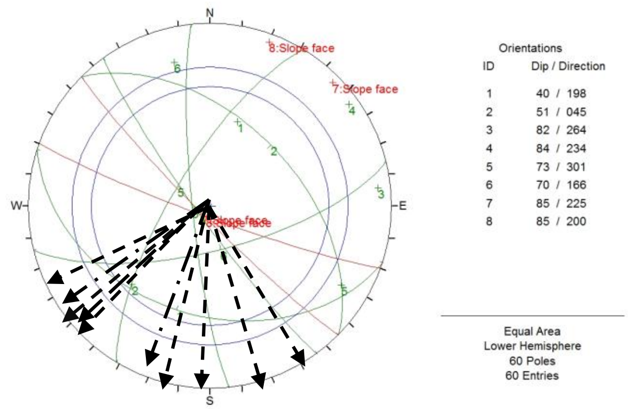

Fig. 5. Stereographic projection of the joints sets along the northeastern side of the slope, show the failures directions.

Table 1. Modes of failures of the rock slope north east side.

Slope face ID #	Failure along joint set #	Type of failure	Direction of failure
7&8	4 and 1	Plane	233 & 198
8	2	Toppling	224
7&8	1&5, 6&5, 6&3, 6&4, and 1&3	Wedge	224, 231, 193, 160, and 180
7&8	1&6 and 1&4	Wedge at wet condition	242 and 147

The deterministic analysis was applied using the *RocPlane* computer program, to draw the three-dimension and side view graphs of the rock slope (Fig. 6). It should be noted that the seismic coefficient was given various values as 0, 0.1, 0.2, 0.3, and 0.4. The input data collected from the lab. and field tests are as follows: slope angle 80°, slope height 8m, tension crack angle 85°, upper slope face 35°, mi 13, mb 1.15, s 0.0005, a 0.52, USC 7,440 (t/m^2), GSI 15, 22, and 30, unit weight 2,760 (kg/m^3), rock blots 8m, rock bolt angle 15°, rock bolt capacity 20 (t/m) (Table 2). The percentage of water filling the tension crack was assumed to be 100% as a conservative value in the process of calculating the factor of safety.

The factor of safety equals to 0 at this station (Table 2). However, after performing the support analyses by adding rock bolt of 20 ton/m capacity (Fig. 7) at seismic coefficient (Sc) = 0, the factor of safety reaches up to 6.02, and decrease to 4.57 as the seismic coefficient increases up to 0.3, showing a negative trend relationship (Table 2). Finally, at seismic coefficient equals 3.75, the factor of safety start to sharply decreases towards 0, i.e. complete failure, at seismic coefficient equals 0.4 (Fig. 8). This means that the support capacity at 20 ton/m will not stand that seismic event strength, more tension should be applied to withstand seismic coefficient more than 0.4.

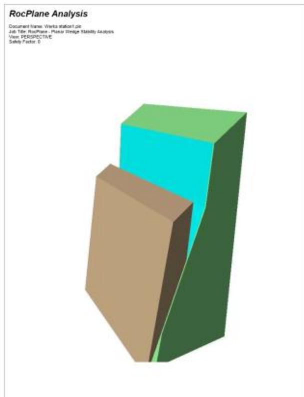

Fig. 6. The 3D plot of the plane failures north east side, similar to the actual failure shown at Fig. 4.

Table 2. The lab. and field data required for the plane failure analyses at north east side of the slope.

	Sc = 0	Sc = 0.1	Sc = 0.2	Sc = 0.3	Sc = 0.4
parameter	FS	FS	FS	FS	FS
Rock bolt length (m)	6.02	5.51	5.03	4.57	0

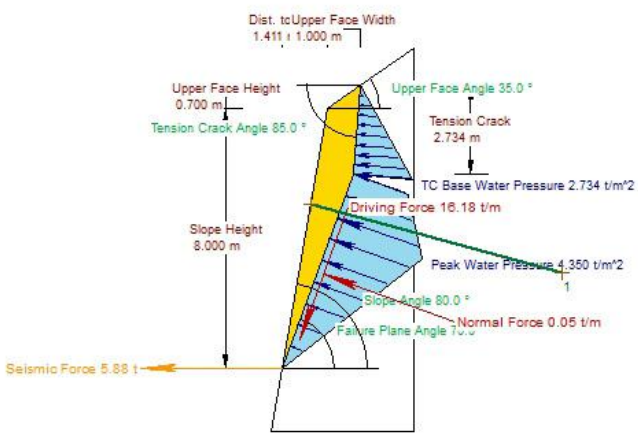

Fig. 7. A side view of the forces acting on the plane failure.

<u>*Wedge failures modeling at north east side*</u>

Modeling of the wedge failures (Figs. 4 and 9) show that it is possible to take place. Graphical modeling of the joints sets indicates possibility of many wedge failures to occur (Fig. 5).

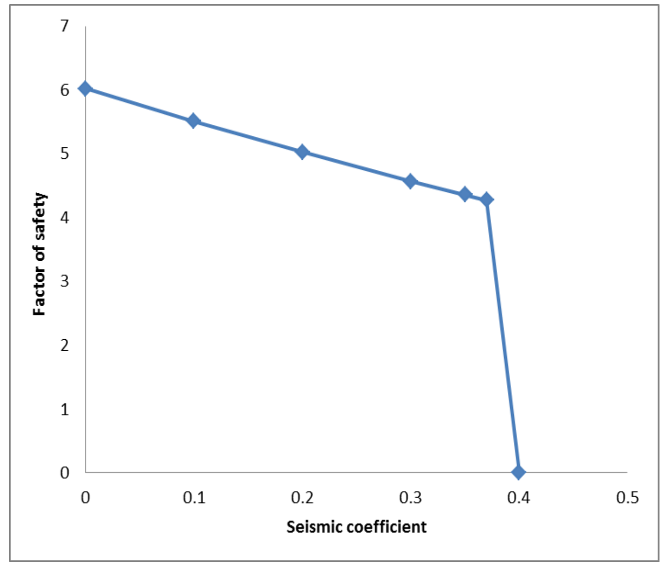

Fig. 8. The negative relationship between the factor of safety and seismic coefficient, where failure occurs at 0.4 seismic coefficient.

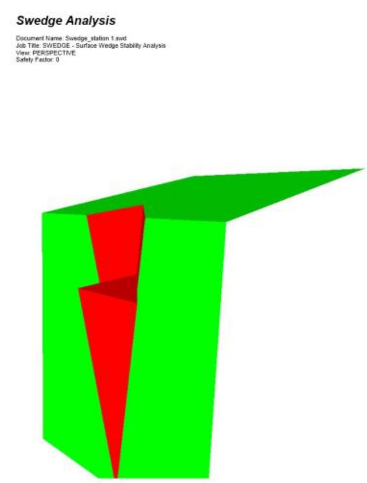

Fig. 9. The 3D plot of one of the wedge failures northeastern side, similar to the actual failures shown at Fig. 4.

The factors of safety of the wedge failures vary in a wide range (Table 3) before and without support. Modeling of the effect of the seismic coefficient (Sc) shows that the factor of safety varies at each coefficient. At seismic coefficient = 0 after support, the factors of safety are generally more than 1 (Fig. 4). After support by anchor length 8.7 m, bolt trend 45, plunge and capacity 10°, 50 (t/m), and at seismic coefficient = 0.1 after support, some of the factors of safety decreased (Fig. 5). The factors of safety decrease to <1 at seismic coefficients 0.2 and 0.3, Tables (4, 5, 6, and 7). These results indicate that the factors of safety decrease as the seismic coefficient decrease.

Table 3. Wedge failures factors of safety along northeastern rock slopes, before support, at various seismic coefficients.

Slope face ID # 7&8	Factor of safety			
	Sc = 0	Sc = 0.1	Sc = 0.2	Sc = 0.3
1&5	1.02	0.88	0.77	0.68
6&5	0	0	0	0
6&3	0	0	0	0
6&4	0	0	0	0
1&3	0.71	0.63	0.56	0.5
1&6 at wet condition	0	0	0	0
1&4 at wet condition	0.36	0.34	0.32	0.3

Table 4. The factors of safety for the wedge failures along northeastern rock slopes, after support, and Sc = 0.

Factors of safety at joint sets						
1&5	6&5	6&3	6&4	1&3	1&6	1&4
1.14	1.59	1.43	11	0.84	2.8	0.6

Table 5. The factors of safety for the wedge failures along northeastern rock slopes, after support, and Sc = 0.1.

Factors of safety at joint sets						
1&5	6&5	6&3	6&4	1&3	1&6	1&4
0.97	1.39	0.84	8.95	0.73	2.14	0.56

Table 6. The factors of safety for the wedge failures along northeastern rock slopes, after support, and Sc = 0.2.

Factors of safety at joint sets						
1&5	6&5	6&3	6&4	1&3	1&6	1&4
0.84	1.23	0.79	7.54	0.65	1.82	0.52

Table 7. The factors of safety for the wedge failures along northeastern rock slopes, after support, and Sc = 0.3.

Factors of safety at joint sets						
1&5	6&5	6&3	6&4	1&3	1&6	1&4
0.75	1.1	0.74	6.52	0.58	1.59	0.48

<u>Plane failures modeling at south east side</u>

The great circles of the prevailing joints sets show that the plane failures (Fig. 10) could take place, as they are close to the friction angle value at wet conditions. The plane failure results are given at Table 8.

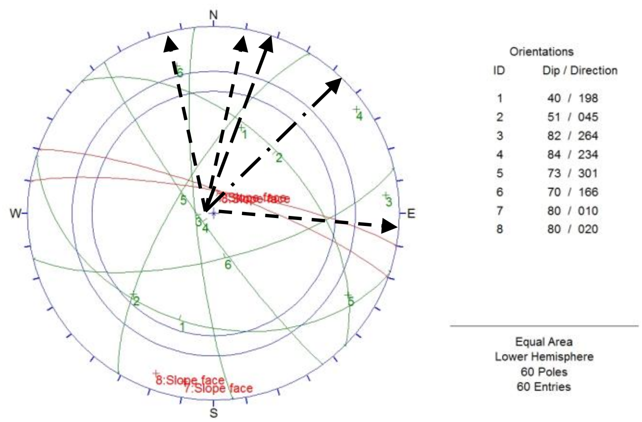

Fig. 10. Stereographic projection of the joints sets along the southwestern side, show the failures directions.

Table 8. Modes of failures of the rock slopes, south west side.

Slope face ID #	Failure along joint set #	Type of failure	Direction of failure
8	2	Plane	45
7&8	1	Toppling	18
7&8	2&5	Wedge	12
8	2&6	Wedge	93
7&8	2&3	Wedge at wet condition	348

The deterministic analysis was applied using the *RocPlane* computer program, to draw the three-dimension and side view graphs of the rock slope (Fig. 6). It should be noted that the seismic coefficient was given various values as 0, 0.1, 0.2, 0.3, and 0.4. The input data collected from the lab. and field tests are as follows: slope angle 85°, slope height 8m, tension crack angle 85°, upper slope face 15°, mi 13, mb 0.63, s 0.0008, a 0.56, USC 7,440 (t/m^2), GSI 15, 22, and 30, unit weight 2,760 (kg/m^3), rock blots 8m, rock bolt angle 15°, rock bolt capacity 20 (t/m) (Table 9). The percentage of water filling the tension crack was assumed to be 100% as a conservative value in the process of calculating the factor of safety.

The factor of safety equals to 0 at this station (Table 9) at wet condition. However, after performing the support analyses by adding rock bolt of 11 ton/m capacity (Fig. 11) at seismic coefficient = 0, the factor of safety reaches up to 1.99, and decrease to 1.22 as the seismic coefficient increases up to 0.3, showing a negative trend relationship (Table 9). Finally, at seismic coefficient equals 3.75, the factor of safety start to sharply decreases towards 0, i.e. complete failure, at seismic coefficient equals 0.4 (Fig. 8). This means that the support capacity at 11 ton/m will not stand that seismic event strength, more tension should be applied to withstand seismic coefficient more than 0.4.

Table 9. The lab. and field data required for the plane failure analyses at south east side of the slope.

	Sc = 0	Sc = 0.1	Sc = 0.2	Sc = 0.3	Sc = 0.4
parameter	FS	FS	FS	FS	FS
Rock bolt length (m)	1.99	1.72	1.47	1.22	0.98

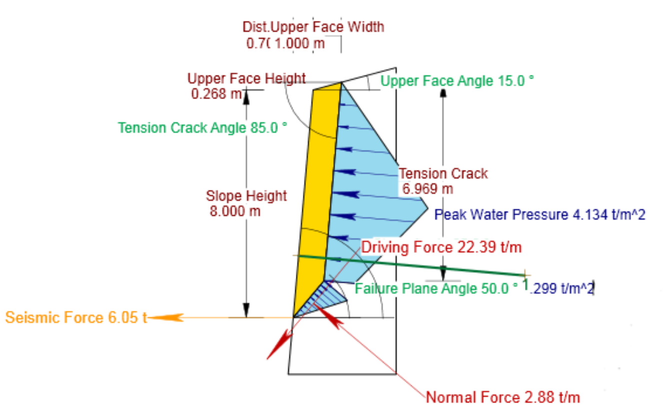

Fig. 11. A side view of the forces acting on the plane failure of rock slope at south west side.

Wedge failures modeling at south east side

Modeling of the wedge failures (Figs. 4 and 12) show that it is possible to take place. Graphical modeling of the joints sets indicates that many wedge failures are possible to take place (Fig. 10).

The factors of safety of the wedge failures are all less than 1 (Table 10) before and without support (Fig. 12). At seismic coefficient = 0 after scaling, the factors of safety are generally more than 1 (Fig. 4). Modeling of the effect of the seismic coefficient shows that the factor of safety increases (Tables 11 to 14). After support by anchor length 8.7 m, bolt trend 45, plunge and capacity 10°, 60 (t/m), and at seismic coefficient = 0.1 after support, some of the factors of safety decreased (Fig. 10). These results indicate that the factors of safety decrease as the seismic coefficient increase. Taking into consideration that the applied support, should be 60 tonnes/meter.

Table 10. Wedge failures and factors of safety along rock slopes at southwestern side, before support, at different values of seismic coefficients (Sc).

Slope face ID #	Failure along joint set #	Factor of safety			
		Sc = 0	Sc = 0.1	Sc = 0.2	Sc = 0.3
7&8	2&5	0.78	0.68	0.61	0.5
8	2&6	0.76	0.7	0.64	0.54
7&8	2&3 at wet condition	0.76	0.7	0.64	0.58

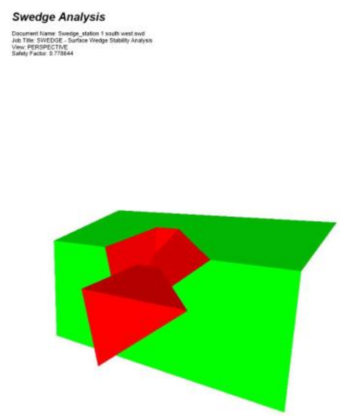

Fig. 12. The 3D plot of one of the wedge failure southwestern side, similar to the actual failures shown at Fig. 4.

Table 11. The factors of safety for the wedge failure analyses along rock slopes at southwestern side, after support, and seismic coefficient = 0.

2&5	2&6	2&3
1.92	2.49	2.49

Table 12. The factors of safety for the wedge failure analyses along rock slopes at southwestern side, after support, and seismic coefficient = 0.1.

2&5	2&6	2&3
1.55	2.06	2.06

Table 13. The factors of safety for the wedge failure analyses along rock slopes at southwestern side, after support, and seismic coefficient = 0.2.

2&5	2&6	2&3
1.3	1.76	1.76

Table 14. The factors of safety for the wedge failure analyses along rock slopes at southwestern side, after support, and seismic coefficient = 0.3.

2&5	2&6	2&3
1.12	1.53	1.53

ROCKFALLS MODELING

Rockfall is a natural result of weathering on steep natural slopes or rock cuts. Rocks falling from steep slopes, natural cliffs, or rock cuts usually travel down the slope in a combination of free fall, bouncing, and rolling. In this scientific report, rockfall refers to rocks traveling in a combination of these modes.

Rockfall presents a common hazard to transportation routes and structures in steep mountainous terrain. Slope material properties influence the behavior of a rock rebounding from a slope. Numerical representations of these properties are termed the normal coefficient of restitution (R_n) and the tangential coefficient of frictional resistance (R_t), where the normal direction is perpendicular to the slope surface, and the tangential direction is parallel to the slope surface (Piteau and Associates, 1980; Wu, 1984).

The triggering zones are located in the upper part of the rock slopes, above the supported slope faces, and characterized by an inclination of 45°-60°. Here debris material, and rock blocks are essentially derived from the upper vallies tributaries and the fragmentation of rock masses, deposits with a very varied grain-size: from few centimeters up to few decimeters (Fig. 13).

The rainfall at this area is so heavy associated with high energy that could move the semi loose and loose rock blocks, which can easily cause damage to the road.

Rockfalls and wash away of loose rocks from the higher elevations were also observed at the descent road, after the rainfall at 22 Dec., 2008 and Jan, 2011, (Fig. 4). More rock blocks were observed resting in a loose condition behind the slope top (Fig. 13) and prone to fall down.

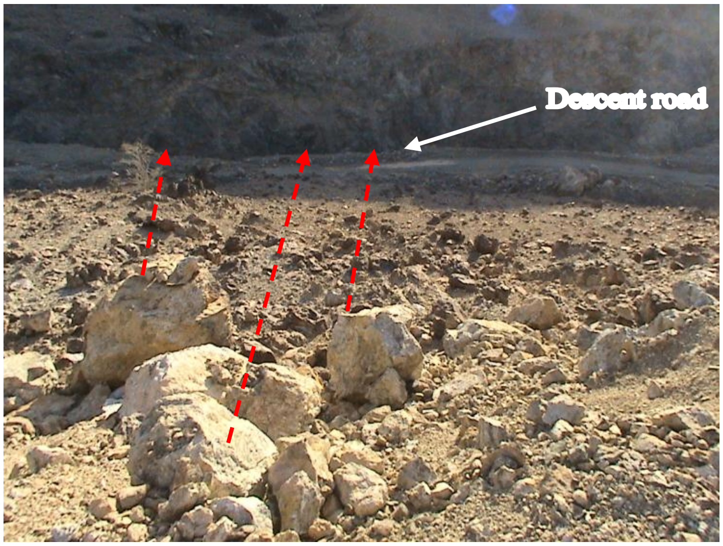

Fig. 13. Loose rock blocks form a rockfall hazard located along the higher elevations along the descent road. Fall directions are shown by arrows.

Rockfall modeling incident is performed by using the computer program *RocFall* (Rocscience, 2010) based on the Pfiffer (1989) concept. The profile of the rock slope where the rockfalls took place shows a number of rock blocks are covering the descent road (Fig. 14). Bounce height of the fallen rock blocks, total kinetic energy, translational velocity, rotational velocity and the 17m end-point are shown below (Figs. 15 to 19). Solution of the rockfalls is by modeling the location of the barriers along the rock slope profile along the higher elevation and the source of the rock blocks, see (Fig. 20).

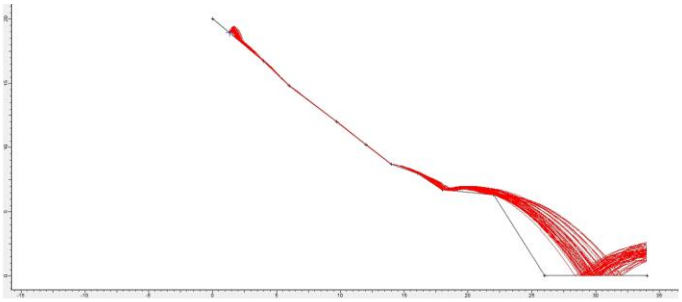

Fig. 14. Modeling of the rockfalls incident along the studied section of the road, as given above in Fig. 13.

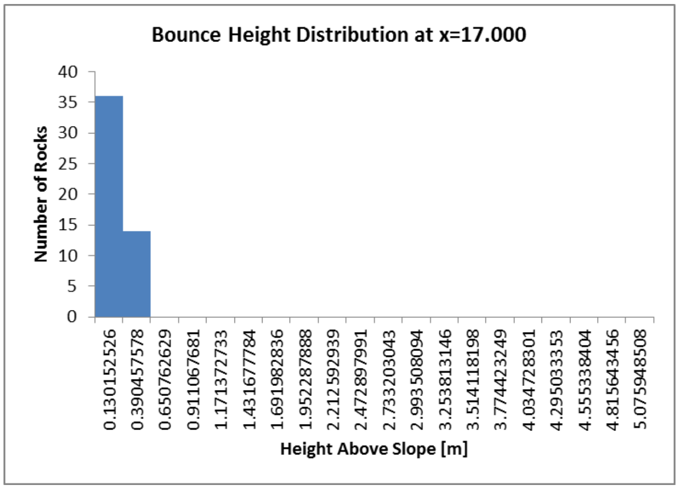

Fig. 15. Bounce height of the fallen rock blocks on the descent road.

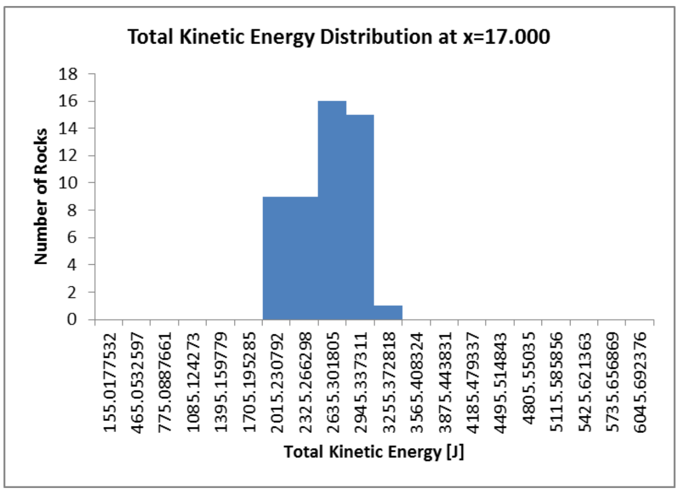

Fig. 16. Total kinetic energy of the fallen rock blocks.

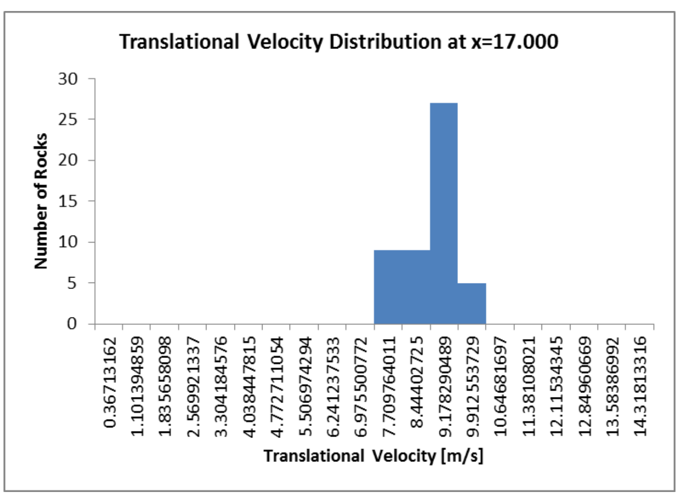

Fig. 17. Translational velocity of the fallen rock blocks.

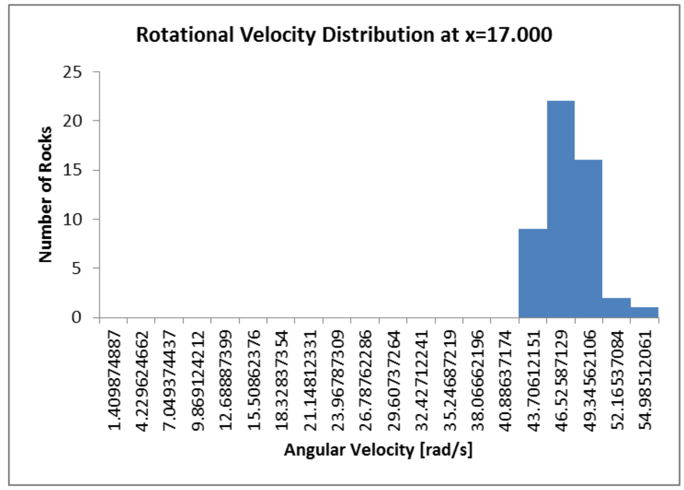

Fig. 18. Rotational velocity of the fallen rock blocks.

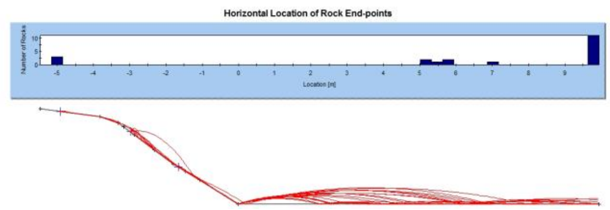

Fig. 19. The end-points of the falling rock blocks, north side.

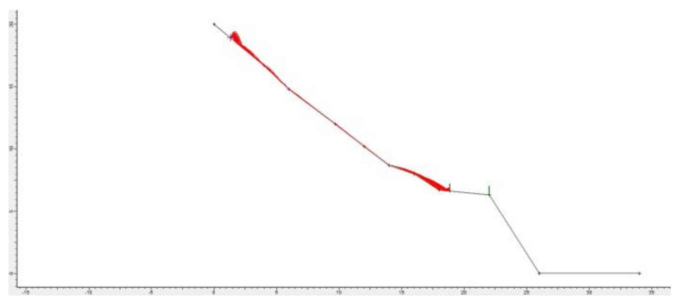

Fig. 20. Modeling of placing of the barriers of 2m height to prevent rockfalls from reaching the descent road.

Modeling indicates the necessity to: 1) the slope redesigns, and 2) to install one vertical 2 m-high rockfall barrier, and one inclined 3 m-high rockfall barriers along and above the slope bench (Fig. 20). The mesh should have a capacity of >10 kj higher than the modeled kinetic energy of the rockfalls in order to stop the falling rock blocks from reaching the road. The total preserved kinetic energy after placement the modeled barrier is 0 kj (Fig. 21). Modeling of bounce height = 0 m (Fig. 22) proves that solution.

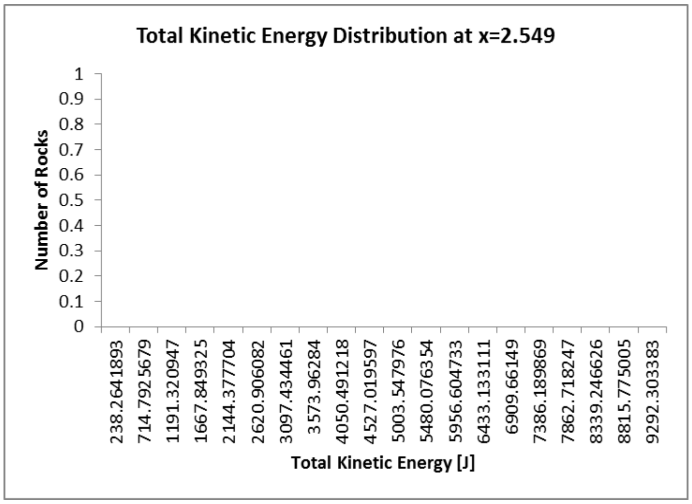

Fig. 21. Modeling of the total kinetic energy after placement of the modeled barriers to prevent the falling rock blocks from reaching the road, north side.

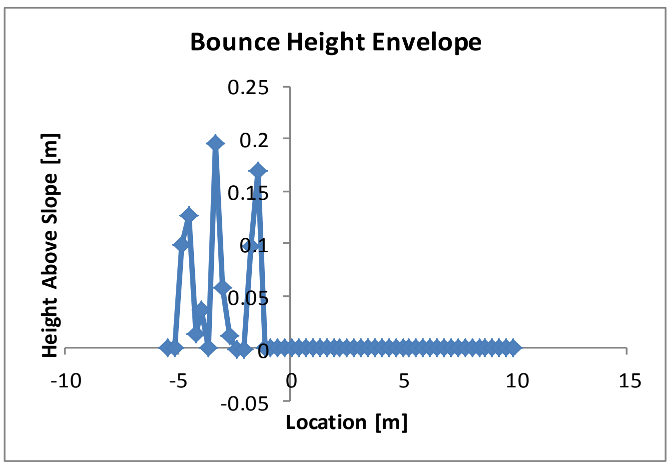

Fig. 22. Modeling of the bounce height after placing the modeled barriers to prevent the falling rock blocks from reaching the road north side.

CONCLUSIONS

1. The integrated techniques, including field and laboratory testing programs, *RMR* and *GSI* classification systems, *GPS* surveys, satellite data, and recent software packages of *DIPS* and *RocFall* covered the necessary data for the stability and remedial measure's requirements.
2. Rock slope failures and rockfalls, and debris flows from upper steep elevations, occur frequently along the Werka descent road, mainly during the rainy seasons harm the road.
3. The kinetic energy at the sites is mainly translational, as the slopes are very steep. Accordingly, the slopes should be redesigned to make benches.
4. As the steep rock slope face angle increases the translational kinetic energy of the rock falls, and become equal to the total kinetic energy.
5. Barrier meshes should have a kinetic energy capacity higher than the translational capacity of the fallen rock blocks, in order to prevent it from reaching the road.
6. Mesh barriers aimed to stop the rock falls from reaching the road. However, it is not the only remedial measure to be taken at the slopes. Shotcrete, drape, rock bolts are also suggested to be taken.
7. Ditches along the rock slope faces should be made to collect the rockfalls.
8. The analyses indicate that the intensity of rainfall, joints set attitudes with the slope face attitude, J_v, block size, block shape, specific gravity, coefficients of restitution and the slope geometry are the main factors in rock slope failure and rockfall problems.

ACKNOWLEDGMENT

The author expresses his gratitude to the King Abdulaziz City for Science and Technology KACST for supporting this research project under grant # AT-29-305.

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