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Rock Slope Stabilization and Protection for Roads and Multi-Storey Structures in Jabal Omar, Saudi Arabia

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Abstract—Jabal Omar is located in the western side of Makkah city in Saudi Arabia. The proposed Jabal Omar Development project includes several multi-storey buildings, roads, bridges and below ground structures founded at various depths. In this study, geological mapping and site inspection which covered pre-selected areas were carried out within the easily accessed parts. Geological features; including rock types, structures, degree of weathering, and geotechnical hazards were observed and analyzed with specified software and also were documented in form of photographs. The presence of joints and fractures in the area made the rock blocks small and weak. The site is full of jointing; it was observed that, the northern side consists of 3 to 4 jointing systems with 2 random fractures associated with dykes. The southern part is affected by 2 to 3 jointing systems with minor fault and shear zones. From the field measurements and observations, it was concluded that, the Jabal Omar intruded by andesitic and basaltic dykes of different thickness and orientation. These dykes made the outcrop weak, highly deformed and made the rock masses sensitive to weathering.

Keyword—Rock, slope, stabilization, protection, Makkah.

I. Introduction

THIS study presents the rock slope stabilization and protection measures that may be required after the blasting works to re-profile the slope. The methods of stabilization of rock slopes generally fall into three different categories; reinforcement, rock removal and protection.

The application of these proposed stabilization and protection measures is very much dependent on the rock slope conditions after blasting works. Geological and geomorphological mapping would have to be undertaken immediately after the blasting works to identify potential localized failures.

II. Materials and Methods

Due to time constraint, the geological mapping and site inspection was done in a quick walk-over survey and only covers pre-selected critical areas which are accessible by foot [2], [8]. In the walk-over survey, the pertinent geological features such as lithological types, structural discontinuities, degree of weathering, and the existing and anticipated geotechnical problems were noted and recorded. Photographs of the relevant geological and geotechnical features were also captured for the record.

In this study, degree of weathering for the rock masses is described using the classification scheme. Some of the terminology used to describe the discontinuity features and field-estimation of the rock strength are illustrated.

III. Results

A. Geology of the Site

The geology of the site consists predominantly of fine to medium-grained, dark grey to greenish grey andesite. The andesite is locally intruded by dolerite dykes of various thicknesses which are ranging from 0.1 m to l.5 m thick. The dykes are generally intruded into the pre-existing shear and/or fault zones [2], [7].

The exposed rock mass varies from moderately to highly weathered (grade III to grade V) rocks [4]-[6]. Better grade and fresher rock mass is expected to occur at a deeper level. Zones of highly to completely weathered rock mass is generally encountered a few (3-Sm) meters below the original topography and within these zones the rock masses are characteristically weak to very weak. Degree of fracturing generally much more intense in these zones which due to the effect of stress release from the demolition of the previous buildings and removal of the overburden [2].

In general, the rocks are heavily jointed, faulted and sheared due to tectonic deformation [7]. Along the shear and fault zones, the rocks are commonly highly fractured and sheared with some degree of alteration and are highly weathered.

B. Existing Site Condition

The site was selected as a critical area since the exposed bedrocks by recent rock cutting are generally in very poor condition, heavily jointed and sheared. Based on information from the Resident Engineer on site, rock slope failures occurred recently at the upper portion of the cut slopes which covered by "concrete spray" (Fig. 2).

C. Geotechnical Problems

The geotechnical problems encountered at this site are mainly related to rock slope instabilities due to poor rock mass condition. The rock mass in this area is riddled by at least 5 to 6 sets of discontinuities, mainly in the form of joints, faults, and shear zones. The upper part of the bedrock mass is also highly weathered. Intersection of these discontinuities resulted in loose blocks of various shapes and sizes, which often

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inhibits the creation of open, smooth vertical rock cuts [3]-[5]. This is evident in the field where in many places, fallen wedges and blocks can be found along the foot slope and their scars can be clearly observed in the slope faces.

In view of the danger and risks of rock falls to the people at work and their machineries, the earthwork contractor has applied "concrete spray" as temporary measures to protect the slope prior to advancing their excavation works to a deeper level [11]. However, this protection measure was proven unsuccessful. Wedge failures, rock falls, and tension cracks had since developed, suggesting that the "concrete spray" was inadequate and could not support the vertical cut in poor rock mass and thus some other slope protection measures have to be adopted. From a brief survey, it was also found that the cut slopes along the site Mare potentially unstable and are very likely to fail either in the form of wedge, toppling and/or rock falls. Some of the identified critical spots are shown in Fig. 3.

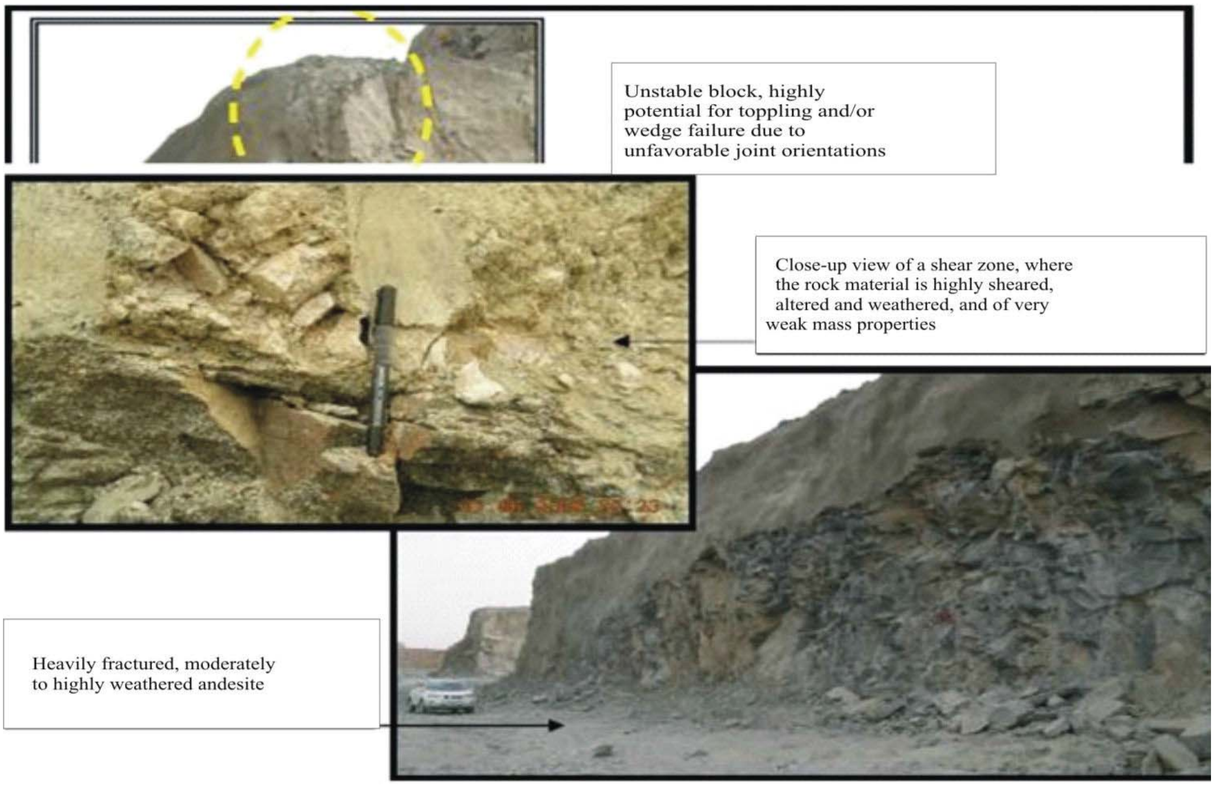

Fig. 1 Some of the common geological features of the rock mass along the Site

Fig. 2 View of the site condition of the site, the yellow circle indication the failure section

D. Discontinuity Survey & Kinematics Stability Analysis

Geological mapping works was carried out in the vicinity of the unstable slope shown in Fig. 3 (a) in order to assess and to verify its stability. The discontinuity data orientation was plotted into lower hemisphere stereographic projections and analyzed (Fig. 4). The results of the analysis (Fig. 5) confirmed the presence of unstable blocks, which is defined by the intersection of three (3) major sets of joints (H: 340/35; J2: 322/90, and B: 256/78). The toppling failure is made possible by the intersection of the steeply dipping J2 with plane H acting as the basal sliding plane. The failure may also take place in the form of wedge due to the intersection of J3 and J1 [6] and [9].

The study shows that;

- Geology of the area along the site consists of moderately to highly weathered, heavily jointed, fine-grained andesite which has been variously intruded by dolerite and basaltic dykes, notably along major shear & fault zones.
- At least 5-6 sets of major joints can be dictated from the rock cuts. The joint spacing is generally very low, notably approaching major faults and shear zones.
- Poor blasting techniques further aggravated the rock mass quality, which is already naturally of very low quality due to tectonic deformation (intense jointing, shearing and faulting).
- Existing and potential modes of failure includes:
- Planar failure rock slide
  - ➢ Wedge failure
  - ➢ Toppling and rock/boulder falls

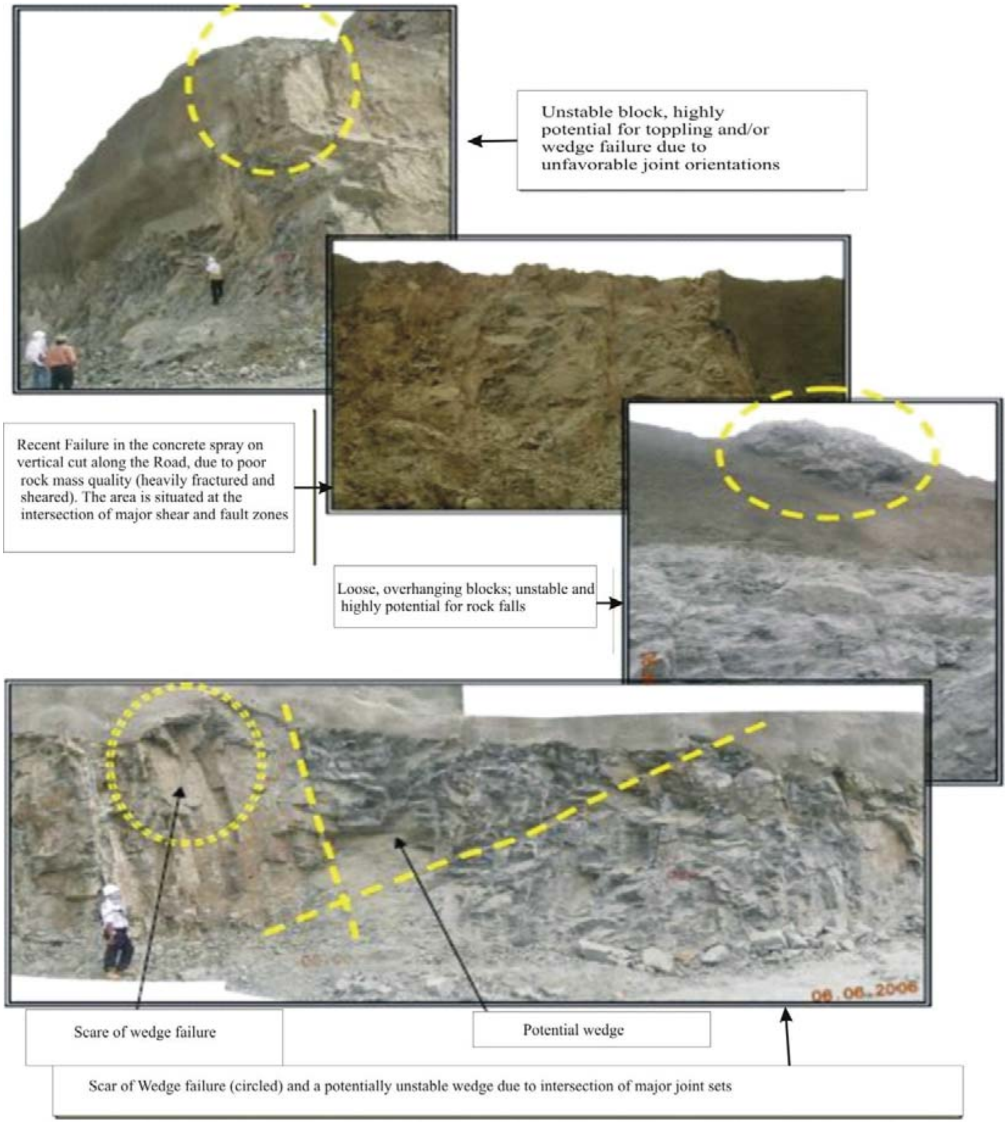

Fig. 3 Examples of some of the identified geotechnical problems associated with vertical cut in poor rock mass along the site

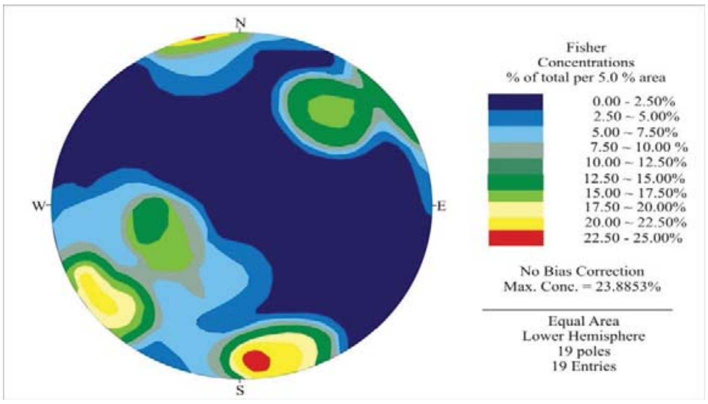

Fig. 4 Stereographic plot the discontinuities of the failed slope

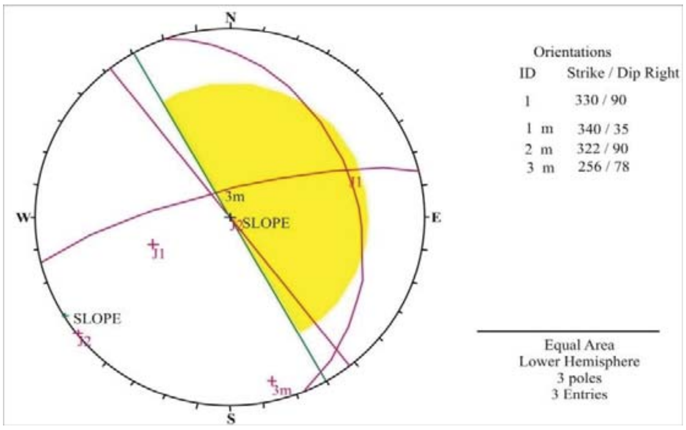

Fig. 5 Kinematic stability analysis of the rock slope at the site, Sta. 0+160

E. Wedge Failure Analysis

The wedge failure identified from the kinematics stability analysis (Fig. 5) was further modeled by using SWEDGE v4 of Rocscience in order to determine its factor of safety during the failure. In this back analysis, it is assumed that the friction angle along the discontinuity plane is 36° and the failure took place in dry condition. Results of the analysis (Fig. 6) suggested that the factor of safety was 1.07.

Detail wedge failure analysis was also carried out in order to determine the best possible methods of stabilization (Fig. 7). Patterned-bolting was proposed as a stabilization measures. The analysis results suggest that the wedge could be stabilized and its factor of safety can be increased to 1.5069 by installing at least 12m long, Y32 bars with capacity of 30 tons of rock bolts.

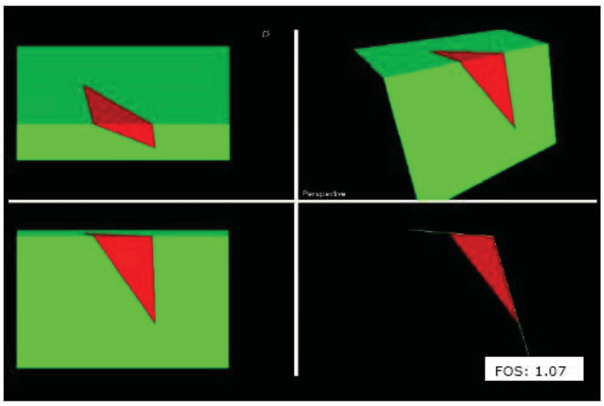

Fig. 6 Back analysis modeling of the unstable wedge, by using SWEDGE v4 program

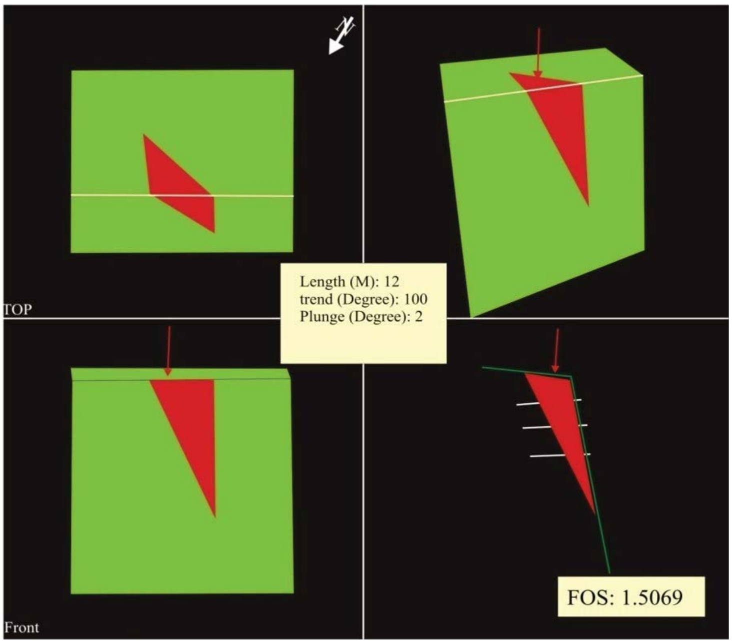

Fig. 7 Detail analysis to determine suitable length of the rock bolts

Fig. 8 The proposed mitigation measures for the unstable slope along the site - the concrete-covered upper part of the slope to be removed, supported with MSE wall and backfill with suitable fill materials

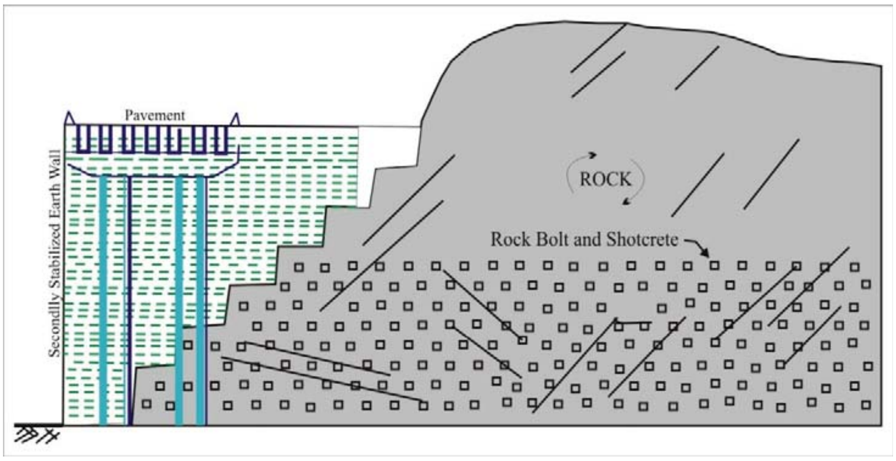

Fig. 9 Schematic diagram showing the proposed stabilization measures for the rock slope along the site

IV. Conclusion

Long term stability of the site is vitally important in order to achieve the intended design profile [1], [10]. However, the natural ground conditions with poor rock mass quality (heavily jointed, faulted, and sheared) hindered the creation of steep vertical cuts in the rock mass. The following measures are recommended:

- The upper part of the slope, which is highly weathered and heavily jointed, is proposed to be removed because the existing "concrete spray" cover has hindered detailed mapping of the geological structures, identification of the critical areas and proper selection of suitable protection/stabilization measures (Fig. 8).
- The mechanically stabilized earth wall such as VSL, Nehemiah or RE Wall can be applied to create vertical wall and to maintain the original platform design for site. The wall should be founded on sound rock and this has to be identified by the geologist on site. The wall is to be tied back to the slope prior to backfilling with suitable materials (Fig. 9)
- Shotcrete - the shotcrete shall be reinforced with steel wire mesh and tied back to the slope with rock dowels/bolts.
- Rock bolts - Patterned-bolting can be applied to the heavily jointed areas.

Spacing and length of the patterned-bolting should base on the results of detailed slope mapping by an experienced geologist. Passive rock bolts are preferred because the nature of the heavily fractured/jointed rock mass is unfavorable for application of active rock bolts. Combination of these stabilization and protection methods is schematically summarized in the cross-section along the site as shown in Fig. 9.

The failed section below the proposed bridge abutment along the site has to be trimmed and stabilized with patterned-bolting in combination with shotcrete. The results of this study indicated that the rock slope for the foundation of the bridge abutments is unstable; the proposed bridge at the site shall be designed on a stand-alone structure (bridge pier) as shown in Fig. 9. The structure should be founded on sound bedrock.

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