Slope Stability with Permanent Rock Anchors

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ABSTRACT

This paper presents the design and construction of permanent rock anchors for the stability of rock slope in Makkah, Kingdom of Saudi Arabia. The project site was located over the steep hillside along the Jabal Kaaba Road. The height of the slope was 58 m.

The geology of the area surrounding the site was mainly composed of granodiorite rocks. Structurally the rock formation was affected with various discontinuities and local faulting. Both field and laboratory studies were carried out. The field study involved detailed discontinuity surveys. Based on the information collected in the field and laboratory, the slope stability of granodiorite was investigated. The kinematical, limit equilibrium and numerical analysis were carried out to determine slope stability. The extensive rock anchor programme undertaken as part of the rock slope stability required the formation of double corrosion protected permanent anchors in weathered granodiorite formation.

INTRODUCTION

The Jabal Al Kaaba project in Makkah, Kingdom of Saudi Arabia was the staged development of a total area of 46700 m$^2$ containing a complex of several hotels. In order to maximize the exploitable land area against the volume of excavation, the use of near vertical cut faces with no berms was required. The development required a large excavation to be blasted into the side of granitic hill. This would result in the formation of vertical cuts of up to 58 meters in height.

SITE CONDITIONS

Surface Conditions

The project site was located over a steep hillside, known as Jabal Al Kaaba, along Jabal Al Kaaba Road, in the city of Makkah Al Mukarramah. Elevation over the site area varies from approximately 303 m at the street level to 350 m at the top of the hill (Dar Al-Handasah, 2002).

A 35 m diameter, 3000 m$^3$ capacity water reservoir exists immediately adjacent to the crest of highest cut face. Another rectangular water reservoir (Al Hafayer) was located close to the crest of the proposed cut face. The vertical excavation was extended to elevation +290 m and formed a vertical cut of 58 meters in maximum height. The height of the cut at Al-Hafayer reservoir was 33 meters (Figure1).
Subsurface Conditions

Information regarding the subsurface conditions across the site area was obtained by drilling of four boreholes. The locations of the boreholes are shown on Figure 1. A subsurface profile showing subsurface conditions is shown on Figure 2.

The encountered subsurface materials can be classified as follows:

**Fill:** It consists of fine to coarse, angular, silty/sandy gravel with cobbles and was encountered up to 4.5 meters depth in borehole no. BH-1, located at street level, along Jabal Al Kaaba road. This area was previously occupied by demolished residential buildings. Excavated/graded down bedrock, in the form of angular, fine to coarse, Silty/sandy gravel with cobbles and boulders was encountered in borehole no. BH-2A and BH-3, up to 1.0 meter and 1.2 meters, respectively. No fill material was encountered in borehole BH-2 where bedrock was exposed at surface.

**Bedrock:** Granodiorite bedrock was encountered below the fill material at depth varying from 1.0 meter to 1.2 meters in borehole BH-2A and BH-3 respectively and at 4.5 meters depth in BH-1. In borehole BH-2, bedrock was encountered at surface.

Bedrock encountered in borehole BH-2 was characterized as slightly weathered to fresh, strong to very strong, close to medium jointed. Total core recovery (TCR) ranged from 87% to 100%, while solid core recovery varied from 46% to 100%. Rock Quality Designation (RQD) ranged from 46% to 100%.

In borehole BH-2A, bedrock encountered was characterized as moderately to highly weathered, moderately weak to moderately strong, very closely jointed and fractured diorite. From 6.50 meters depth, the bedrock grades to highly to completely weathered granodiorite/diorite. Between 1.0 meter and 10.5 meter depth, TCR varies from 73% to 100% while solid core recovery varied from zero to 69%. RQD ranged
from zero to 59%. From 14.0 meters, bedrock grades to moderately weathered, moderately strong chloritic rock having close to medium joints and fractures. From 20.5 meters to 25.2 meters bedrock is slightly to moderately weathered, moderately strong diorite.

From 25.2 meters to bottom of the borehole at 63.0 meters, bedrock is slightly to moderately weathered, strong granodiorite. From 10.5 meters to the investigated depth of 63.0 meters, TCR was 100%, while solid core recovery and RQD ranged from 33% to 100% (URS/Dames and Moore 2001).

Figure 2. Subsurface profile

Groundwater

Perched groundwater was encountered in BH-2A at a depth of 26.0 meters. Water movement was controlled by the fractures and joints in the bedrock.

Geology

Precambrian granodiorites-diorites which belong to the Arabian Shield crop out in the project area. This unit, which constitutes the bedrock in the project area, is overlain by talus formed of 1-1.5 m thick residual soil and transported material. The granodiorites-diorites are cut by frequent felsite veins and diabase dikes. The characteristics of these units are given below:

Granodiorite-Diorite: This unit has an igneous rock origin and is gray, dark gray in colour, medium-coarse grained, slightly to moderately weathered, and strong to very strong. Jointing has randomly developed in three directions. Weathering grades into slightly weathered stage after the depth of 10-12 m and it becomes completely fresh below the depth of 25m.
**Felsite Veins:** They are vein rocks in which quartz- feldspar predominates. Felsite veins are light, white, pinkish beige and light gray in colour, 10-30 cm thick, irregular and moderately strong to strong. No weakness zones are observed in their contacts with granodiorites.

**Diabase Dikes:** They are blackish, dark green in colour and have brown weathered surfaces; their thicknesses vary from 15-20 cm to 200 cm. Some of the diabase dikes are very frequently jointed and fractured. 5-7 cm shear zones are observed at their contacts with the bedrock (granodiorite). These shear zones are filled with clay and gouge material. This type of diabase dikes can be qualified as fault zones. Geological cross-section of the project area is given in Figure 3.

**Figure 3. Geological sections showing main discontinuities**

**DESIGN**

**General**

In order to determine the failure mechanisms that could potentially occur in the rock cuttings at Jabal Al Kaaba, the following analytical sequence was performed (Ilker, 2003):

- Characterisation of the intact rock and its discontinuities;
- Characterisation of the rock mass properties;
- Identification of potential sliding blocks and rock mass failure mechanisms;
- Calculation of a factor of safety for each potential slope failure;
- Identification of the critical mode of slope instability; and
- Determine the requirement for slope stabilizing measures.
INTACT ROCK AND DISCONTINUITY PROPERTIES

UCS and Unit Weight

Laboratory tests indicate that the unconfined compressive strength (UCS) of the igneous rocks at Jabal Al Kaaba is highly variable, ranging from 20 MPa to 170 MPa. The weakening effects of weathering are likely to have contributed to this variability in the upper part of the profile, as are the presence of cemented or latent discontinuities. In-situ (wet) unit weights ranged from 26.3 to 29.9 kN/m³ with a typical value of approximately 27.3 kN/m³ (Figure 4). The UCS and unit weight values adopted for design are presented in Table 1.

Table 1. Design UCS and Unit Weight

<table>
<thead>
<tr>
<th>Rock Unit</th>
<th>Depth Below Ground Surface (m)</th>
<th>UCS (MPa)</th>
<th>Unit Weight (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0-10</td>
<td>30</td>
<td>23</td>
</tr>
<tr>
<td>2</td>
<td>10-25</td>
<td>70</td>
<td>24</td>
</tr>
<tr>
<td>3</td>
<td>25+</td>
<td>70</td>
<td>26</td>
</tr>
</tbody>
</table>

Figure 4. Variation of UCS and unit weight with depth
Discontinuity Spacing and RQD

The rock discontinuity information was obtained from the borehole core and mapping of exposed cut faces. Measurements of borehole discontinuities indicate that fracture spacing is generally less than 0.4 m, with little discernable variation with depth. In contrast, RQD increases significantly with depth. The fracture frequency data would suggest that the rock mass quality does improve with depth but not as significantly as the RQD values would imply (According to the International Society of Rock Mechanics recommended methods of rock description (ISRM, 1981), the discontinuities are classified as being closely spaced to moderately spaced.

Discontinuity Dip

Measurement of discontinuity dip relative to the axis of the three vertical boreholes (BH1, BH2A, BH3) shows that the discontinuities are generally shallow dipping, with approximately 70% of structures dipping at angles of less than 300.

Visual inspection of rock cuttings within the general vicinity of the circular water tower demonstrate the abundance of steeply dipping discontinuities oriented parallel to the proposed slopes.

ROCK MASS CLASSIFICATION

Geological and geotechnical investigations carried out at the Jabal Al Kaaba open rock cut excavation area have been taken into account and a rock classification has been established according to Bieniawski’s rock mass classification which is called The Rock Mass Rating (RMR) system of Bieniawski, (1989). The parameters utilized in rock mass classification and accepted corresponding values to these parameters are summarized in Table 2.

<table>
<thead>
<tr>
<th>Table 2. Rock Mass Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
</tr>
<tr>
<td>From</td>
</tr>
<tr>
<td>---</td>
</tr>
<tr>
<td>0.00</td>
</tr>
<tr>
<td>25</td>
</tr>
</tbody>
</table>

ASSESSMENT OF ROCK SLOPE STABILITY

Assessment of slope stability in rocks is usually done through kinematical analyses, limit equilibrium analyses and numerical methods such as finite element method. If the kinematical analysis indicates that the failure mechanism controlled by discontinuities, the stability must be evaluated by limit equilibrium analysis, which considers the shear strength along the failure surface, the effects of pore water pressure
and the influence of external forces such as reinforcing elements or seismic accelerations (Turner and Schuster, 1996).

In this study, the kinematical analysis and finite element method are performed for the upper and lower parts of the rock cut.

**Kinematic Analysis**

Kinematic analyses have been performed for planar, wedge, and rock fall type slides at the Zone I (slightly weathered zone between the depth of 0 m and 25 m) and Zone II (fresh rock zone deeper than 25 m) by utilizing the rock parameters and the properties of discontinuity planes. Kinematical analyses were performed using commercially available software, DIPS 5.0 (Rocscience, 1999). The parameters adopted in the kinematic and stability analyses are given below:

For the depth between 0m-25m  

- $c=50 \text{ kPa}$  
- $\phi = 36^\circ$  
- $\gamma = 23 \text{ kN/m}^3$

For the depth more than 25 m  

- $c=100 \text{ kPa}$  
- $\phi = 45^\circ$  
- $\gamma = 26 \text{ kN/m}^3$

Three major and seven minor joint sets were determined during the site investigations. In this system 70/160, 61/183 and 51/145 discontinuity planes have plane failure potential. In addition to that, most of the discontinuity planes have wedge failure potential. Kinematical analyses of the upper and lower slope are shown in Figure 5.

![Figure 5. Kinematical analysis of upper and lower cuts](image)

Using the results of kinematic analysis, the discontinuity plane 61/183 has been considered as the most unfavorable. Therefore, a planar failure analysis has been done with the Slope/W software (Geoslope, 2004). Bishop and Morgenstern – Price methods were used in stability analysis. The obtained factors of safety are as follows:  

- $FS = 1.061$ with Bishop Method  
- $FS = 0.996$ with Morgenstern – Price method.

Based on the results of stability the analysis, it was concluded that the slope must be reinforced.
Second series of analysis were performed, after reinforcing the rock slope with prestressed anchors of 500 kN that were placed on a mesh of 2m x 1.5m. The obtained factors of safety are as follows: FS = 2.26 with Bishop Method and FS = 2.23 with Morgenstern – Price method. The above safety factors obtained from each method confirm the adequacy of the anchor pattern.

**Finite Element Analysis**

In this study, a two-dimensional finite element program, called Phase² (Rocscience, 2006), was used to analyze the stability of slopes at the site. The planar failure studies have been limited to the weathered granodiorite and the section passing through the water reservoir being the most critical. The design data for the finite element analyses are given in Table 3.

**Table 3. The design data for the finite element analyses**

<table>
<thead>
<tr>
<th>Description</th>
<th>g (MN/m³)</th>
<th>UCS (Mpa)</th>
<th>E (Mpa)</th>
<th>Failure criterion for rock mass: (Hoek- Bray, 1981)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weathered Granodiorite</td>
<td>0.023</td>
<td>40</td>
<td>10,000</td>
<td>m = 2.5  s = 0.004 (good quality, coarse grained igneous rock)</td>
</tr>
<tr>
<td>Fresh Granodiorite</td>
<td>0.026</td>
<td>70</td>
<td>50,000</td>
<td>m = 12.5 s = 0.1 (very good quality, coarse grained igneous rock)</td>
</tr>
<tr>
<td>Dyke</td>
<td>0.023</td>
<td>30</td>
<td>1000</td>
<td>m = 3.5 s = 0.004</td>
</tr>
</tbody>
</table>

**RESULTS OF THE ANALYSIS**

The results of planar failure analysis have been obtained as factor of safety for the slopes without anchors and with anchors. The results of finite element analysis have been obtained in the form of vertical and horizontal stresses in the rock mass, vertical, horizontal and total displacements and the strength factor for each excavation stage. Based on the results of the numerical analysis, the strength reduction factor is obtained as 2.0 between 291.5 and 303 elevations (Figure 6).

Based on the results of slope stability analysis described above, including in situ geological and geotechnical investigations and engineering studies, it was concluded to reinforce the northern slope of the excavated area by permanent prestressed anchors and rock bolts. The prestressed anchors were designed for 500 kN working load and placed on a pattern of 2m x 1.5m. For the upper 5 rows, 35 meter long anchors to be fixed behind the major dyke of 120 cm to 200 cm width.
CONSTRUCTION

Significant part of the Jabal Al Kaaba rock excavation works were completed at the time of design process. The bottom of excavation was close to the elevation 300 m and excavation works were being executed in the west and at the bottom of the excavation area. The remaining part of surface excavation related to the slope, started upon completion of the design. The stability of rock mass was successfully provided under fast track design and construct conditions. Anchor drilling was carried out using crawler drill rigs and skid mounted drill rigs. The upper rows of anchors were drilled using skid mounted drill rigs. On completion of drilling of each hole to the specified depth, the holes were subjected to water test over its whole length. In case Lugeon (Lu) value measured on site was higher than the specified criteria of Lu=1, than the hole was grouted. After initial set, the hole was reamed out and water tested again to meet the specified criteria of Lu=1. Approximately 400 permanent anchors were installed with an average length of 30m. Before commencement of any part of the permanent anchorage works, two trial anchors were installed and tested to ensure that they could carry the specified load with a safety factor of 2.

To protect the slope surface from weathering and avoid the eventual rock falls a shotcrete layer reinforced by two layers of wire mesh was applied. View of rock cut stabilization of the Jabal Al Kaba project is shown in Figure 7.

CONCLUSIONS

The case history presented in this paper illustrates the design and construction of rock cut stabilization completed in Makkah, Saudi Arabia. Based on the information collected in the field and laboratory, the slope stability of granodiorite was investigated. The kinematical, limit equilibrium and numerical analysis were carried out to determine slope stability. The limit equilibrium analysis indicated that
failure was expected without reinforcing the slopes. Shear strength reduction analyses was evaluated using Phase². Based on the results of numerical analysis, strength factor of lower part of slope is obtained as 2.0.

![Figure 7. View of the rock cut stabilization](image)

**REFERENCES**

Rocscience Inc., 2006. Phase² 2 D Finite Element Program for Excavation and Support Design, Toronto, Canada
Canada