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Stability study of rock slope in fractured quartzite at right side of spillway, Middle Marsyangdi Hydroelectric Project, Nepal

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ABSTRACT

A 120 m long and 68 m high rock cut slope is designed at the right side of spillway of Middle Marsyangdi Hydroelectric Project. This paper describes the stability studies performed for the rock cut slopes in jointed quartzite for foundation of spillway.

INTRODUCTION

Nepal has hydropower potential of around 83,000 MW of which about 400 MW has been so far generated. An investigation by Nepal Electricity Authority (NEA) has shown a suitable site for the construction of a diversion dam (Fig.1) with a gross head of 110 m. At a flow of 80 m^3/sec. Middle Marsyangdi Hydroelectric Project expects an average annual energy potential of 70 MW and 398 GWh. The Marsyangdi River gorge at the proposed spillway site is narrow and deep. This, together with nearly vertical joint sets, which are parallel and orthogonal to the gorge, presents a potential risk for toppling and planar sliding, especially in presence of water.

This paper describes design studies in rock cut slope carried out for foundation of the spillway. A construction of foundation involves deep excavation in jointed quartzite. It has to be founded partly on jointed quartzite and partly on treated river material. Especially excavation of the right-bank at spillway area will be done in jointed quartzite rock. The height of the rock cut slope will be up to 68 m. A major part of the cut slope is temporary, but some of it will be permanent. After construction, the remaining space between the spillway and the rock cut slope will be backfilled leaving the upper part of the cut slope exposed. Most rock slope will remain open for approximately two years till the completion of spillway and appurtenant structures.

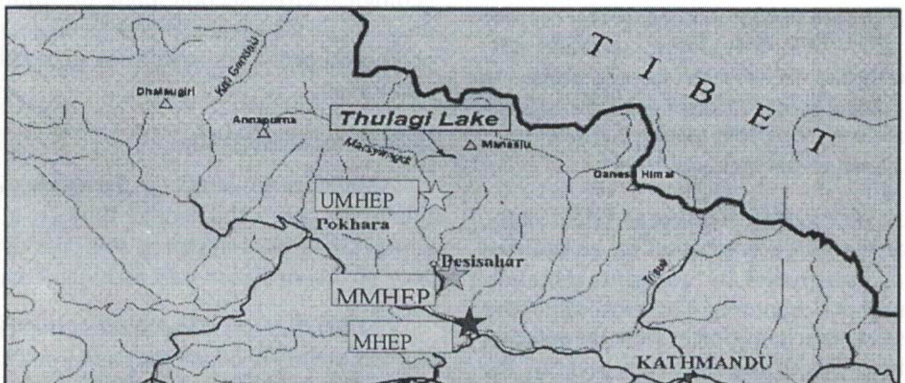

Fig. 1: Location of Middle Marsyangdi Hydroelectric Project (MMHEP) in relation to operating (Marsyangdi) and planned (Upper Maarsyangdi) hydropower stations along Marsyangdi River, Central Nepal.

The designed cut is benched all the way up. Each bench will be 12 m high and shall have an inclination of 8V: 1H. Between each bench, a 3 m wide berm shall be excavated. These will give an approximate overall inclination of 2.5V: 1H, which is equal to 68°. The cut slope was analysed for wedge, planar sliding and toppling failures.

A check for toppling was also carried out in accordance to kinematics of direct toppling. Thereafter, large scale sliding along bedding planes was also analysed.

GEOLOGICAL SET-UP OF PROJECT SITE

The project area is situated in the Lamjung district, approximately 150 km WNW of Kathmandu city. The river Marsyangdi, draining the entire part of the Western Development Region of Nepal, is one of the main rivers of the Sapt Gandaki River system (Yamanaka and Iwata 1982).

Geomorphologically, the project represents the typical geomorphic features of Middle Mountain of Nepal. The designed slope excavation for the right abutment of spillway is proposed at the right side gorge of the Marsyangdi River. The top of the slope is approximately at 630 msl and the bottom of the river is at 560 msl. The existing natural rock slope is almost vertical for 25 m above the riverbed. At the top of the vertical cliff, the valley then changes to a gentle slope of about 20° to 25°. The topsoil consists of mixture of alluvial deposit and humus layer. The thickness of the top layer is approximately 5-10 m.

Quartzite is exposed around the spillway area. The geological map and drill core logs of the right bank slope show the presence of a 100 m thick layered fair to poor quality quartzite with local intersections of faults and shear zones. The quartzite is close to very closely bedded, gently dipping around 16-20°, to the N-NNE. It is slightly to moderately weathered, medium strong to very strong and seamy to massive. The mineralogical composition of quartzite samples from the spillway site shows 93-100% quartz, 2-3% feldspar and 4-7% of mica (Lahmeyer International 1998).

On the basis of air photo interpretation and field study, it was observed that the gorge is situated on an uplifted block of quartzite, controlled by an approximately orthogonal set of faults and fractures. The most important fault is the NNW system, which originates from the splitting of a regional structure, projecting far to the west of the project area. One of its branches runs along the gorge itself, and a second one, runs about 60 m north of the first one. Further, third branch is about 150 m north of the gorge. A second fracture system runs roughly orthogonal to the first one, limiting the sidewalls of the outcropping bedrock. The main joint sets are in N-S and E-W orthogonal pattern, with subordinate diagonal fractures. On the basis of 2D geoelectrical profiling and the interpretation of borehole results, a bedrock contour map was prepared (Fichtner Joint Venture 2000).

The main procedure adopted during the analysis included:

* collection of geological and geotechnical data of exposed discontinuities;
* defining surface roughness and shear strength properties along discontinuities;
* back analysis of the existing natural slope to determine the shear strength parameters;
* identification of representative joint sets and studying them to identify the critical joint sets and combinations;

The main objectives of this study were:

* to optimise the most suitable overall slope angle for a maximum safety factor of 1.10;
* to check the potential of large planar sliding along the bedding plane;
* to perform kinematic analysis such as toppling, small scale planar and wedge failures;
* to optimise the most suitable bench height for a maximum safety factor of 1.05

GEOLOGICAL INVESTIGATION

The geological investigation included surface geological mapping and borehole investigation.

Geological Mapping

Approximately 5–10 m (max) thick alluvial deposit mixed with humus layer overlies the bedrock surface. A typical geological cross section along the spillway axis has been prepared on the basis of investigated data (Fig. 2). No major geological problems have been expected during the slope excavation.

Bore Hole Investigation

Altogether six boreholes have been drilled at the right side of the gorge and another thirteen at the left side of the gorge. Geological mapping and core drilling show fair to poor quality quartzite with local fault and shear zones.

The fill material in the joints at the left side of the gorge consists mainly of minor mica and iron oxide stained clay. In many of the boreholes along the rock spur, some minor band of loam has been detected in the upper part of the boreholes.

In borehole DS 8 at right bank, a relatively thick fracture zone was observed and at the left side of the gorge such type of fracture zones was not found, although drilling was carried out down to 550 msl (Lahmeyer International 1998). In the other boreholes at the right side, many small fracture zones were logged but not as thick as those found at the left side of the gorge. The joint filling at right bank consists mainly of thin mica and FeO. Locally, some clayey filling was

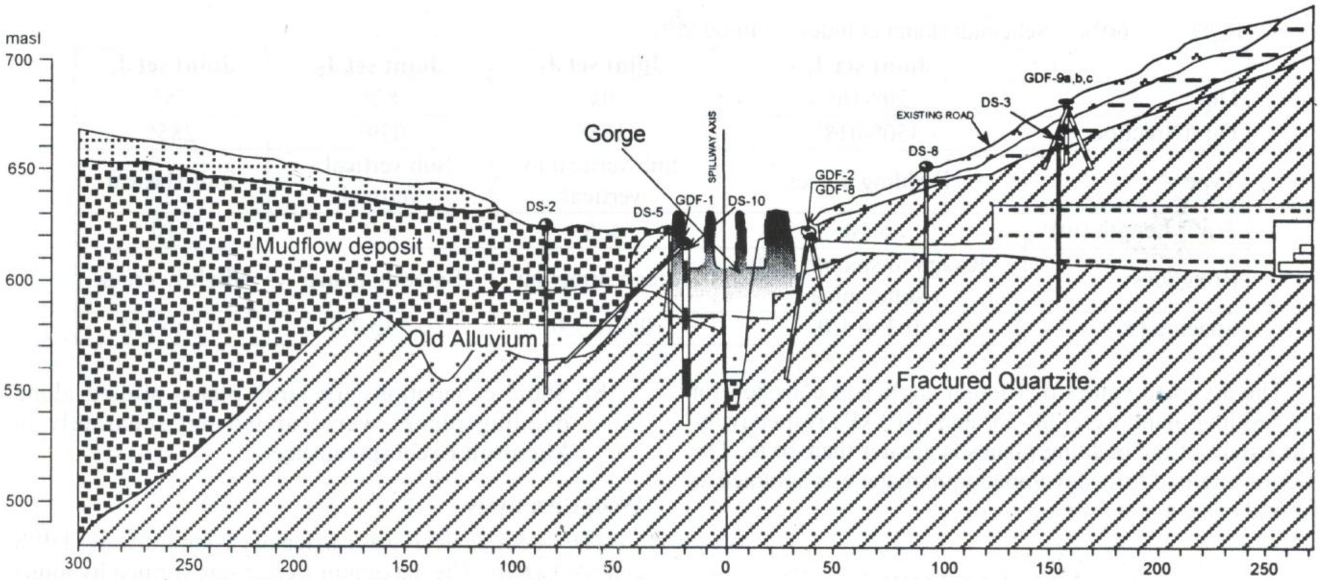

Fig. 2: Geological section through the gorge. In the figure, directly under the spillway one can see the present gorge and to the left of the rock spur the paleo-channels with old alluvium.

found in some joints approximately from 575 msl to the end of the borehole. Surprisingly, thin layer of loam was also found in some joints between 607 to 604 msl in borehole DS-8.

GROUNDWATER

No natural seepage water was observed around the site. According to available records, water table is found around 610 msl at the right bank slope of the river. For the design purpose, ground water level was presumed at 610 msl and descending towards the river water level at approximately 590 msl through fractures. During excavation the ground water level is expected at several meters behind the excavation front.

ENGINEERING GEOLOGICAL MODEL

Table 1 presents the characteristics of the bedding planes and joints at the right-bank side of the gorge (Bieniawski 1989 and Nilsen and Palmstrom 2000).

Load and Dimensions

Back analysis of natural slope was carried out to estimate the overall friction angle. It is common practice to apply earthquake loading to small wedges. For a temporary rock cut, no earthquake load was considered. For the permanent parts of the rock cut, horizontal ground acceleration of 0.21gal. was used.

Intact Rock Strength and Shear Strength Parameters

Laboratory tests have been performed on core samples of quartzite taken from different boreholes. Both uniaxial and triaxial compression tests were carried out to determine the intact rock strength.

The compressive strength from uniaxial and peak and residual strength parameter from triaxial tests were as follows:

Compressive strength

$\sigma_c = 312$ and 355 MPa (failed by bursting)

$\sigma_c = 4.5$, 14.9 and 73 MPa (failed along discontinuity)

Peak and residual strength

$\phi_p = 57.3°$ and $c_p = 49.75$ MPa

$\phi_r = 45.6°$ and $48.3°$ and $c_r = 4.91$ and 10.81 MPa

Point load strength, uniaxial compression and direct-shear test of quartzite from different boreholes were carried out. The uniaxial compression strength calculated from the point load test were as follows:

$\sigma_c = 88.0$ MPa (parallel to bedding direction)

$\sigma_c = 152.9$ MPa (perpendicular to bedding direction)

The uniaxial compression tests show ultimate stress values of 100-160 MPa.

The direct shear tests gave the following shear parameters for the bedding planes:

$\phi = 18.8°, 29.2°, 32.6°$ $c = 0.029 - 0.063$ MPa

Results from these tests reflect the lowest friction angle obtained for a coated joint. The basic friction angle for unfilled joints is according to literature around 30°.

Table 1: Engineering Geological data of bedding plane and rock joints

RQD: ~60% Schemidt Hammer Index: 40-60 MPa

	Joint set J_1	Joint set J_3	Joint set J_2	Joint set J_4
Dip	20°-16°	90°	82°	75°
Dip Direction	350°-018°	185°	039°	285°
Type	Bedding planes	Sub vertical to vertical	Sub vertical - vertical	Sub-vertical
Trace Length (m)	> 10	> 6	> 3	<10
Roughness	Rough to smooth with some mica	Rough, irregular	Smooth, planar	Slightly rough
Spacing (m)	0.2 to 1.0	0.5 to 2.0	0.5 to 2.0	0.5 to 2.0

The assigned values of friction angle and cohesion for the bedding plane were 28^0-35^0 and 0-10 kPa respectively. The extent of the bedding planes is greater than 10 m. The friction angle of fully developed joint plane was assumed as 40^0 and cohesion as 0 kPa.

SAFETY FACTORS

As has been proposed in the German Standard DIN 4084 (1981) "Calculation of Terrain Rupture and Slope Rupture" the required safety factor should take the size and the probability of sliding of the investigated rock mass block into account. The guidelines are given in Table 2.

The friction angle for the bedding plane is one of the most important material parameters. It has been determined by back-calculating the factor of safety of the natural rock slope. For a safety factor ≥ 1.0 without considering seismic load, the average friction angle along the bedding plane was determined as 29.5°. Considering a horizontal pseudo-static seismic load of 0.21 gal the friction angle would have to be at least 33.5° to prevent sliding.

Table 2: Proposed Global Rock Mass Sliding Safety Factors

Block No.	Width	Height	Safety Factor
1-2 and 1 - 3	10-20 m	20-40 m	1.40
1 - 4	20-25 m	50-55 m	1.30
1 - 5	25-30 m	50-55 m	1.20
1 - 6	30-35 m	50-55 m	1.10

STABILITY ANALYSIS

The stability analysis has been performed in two steps and with two different methods.

a) Kinematic analysis was carried out for the identification of local wedge failures, toppling and planar sliding. The analysis has been carried out using commercially available software DIPS (Version 5) and wedge analysis by SWEDGE (Version 4) developed by Rocscience, Inc Canada.

b) Static calculations for large planar sliding along bedding planes were carried out separately in Microsoft Excel spreadsheet.

Wedge Analysis

The wedge analysis was carried out using softwares DIPS and SWEDGE. The maximum wedge size formed by joints $82^0/039^0$ (J_2) and $75^0/285^0$ (J_4) was taken into consideration for the estimation of capacity of rock bolts and the bolting pattern. This means that the inclination of each bench was designed to give the minimum size of the wedges. In the bench slope 8:1(V:H) one gets wedges weighting of 32 ton each and a factor of safety against sliding of 0.5. The wedge will have an exposed area of around 26 m^2. A change in the dip angle ($16^0 - 20^0$) by $\pm 5^0$ leads to the most unfavourable situation i.e. wedge weight up to 138 ton and has exposed surface of about to 54 m^2. To increase the stability of the smaller wedge, 3 rock bolts (Fig. 3) (working load 100 kN) are sufficient (FOS~2). For the larger one, 8 rock bolts are required (FOS~1.8). The working load of rock bolts shall be 100 kN. The rock bolts are of SN type.

Toppling Analysis

At fresh excavated slope the rock block formed due to different joint sets may not be stable (Fig. 4). For example, when average dip angle for joint set J_3 is 90° or vertical there is no potential for toppling in the cut slope. As there is a scatter of dip angles and dip direction within set J_3, the joints dipping to the south have a potential for toppling (Bilfinger+Berger 2001).

A check of toppling failure was carried out in accordance to kinematic direct toppling. For a stable block, the following rules were applied:

$$\psi < \phi$$

and $$b/h > \tan \Psi$$

whereas, Ψ is the inclination of the bedding, ϕ is the friction angle, b is the spacing of joint set J_3 and h is the height of a bench.

The first equation is fulfilled: 20° < 40°

According to field data, b varies between 0.5 to 2.0 m. For an unfavourable situation it is assumed that the sub-

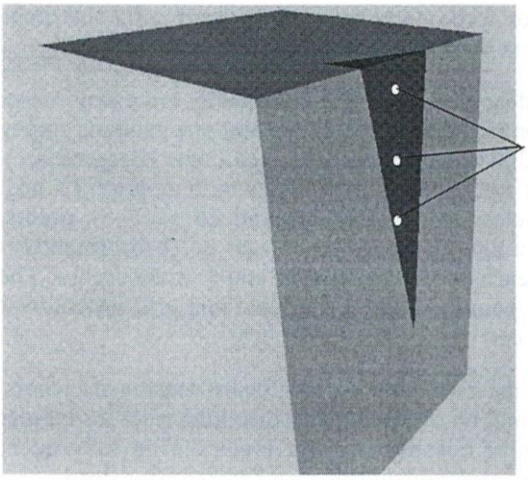

Fig. 3: Wedge analysis

vertical joint trace length is equal to the bench height and block height whereas:

$$0.5/12 = 0.04 < \tan 20° = 0.364 \text{ (unstable)}$$

In this case the block was unstable in the excavated slope. So, the systematic bolting has proposed for stabilization of direct toppling of the blocks.

LARGE SLIDING ALONG BEDDING PLANE

The planar sliding analysis was carried out using DIPS. But the software does not consider joint water pressure in probable failure planes. The large planar sliding along the bedding plane with joint water pressure was considered very serious one. So it seems necessary to carried out the analysis with water pressure. The sliding along a bedding plane at the toe of cut slope i.e. at the bottom of the excavated bench was taken into consideration. The analysis was carried out considering different sizes of potential failure blocks. This analysis was made in two steps:

* Assumption of different friction angles and cohesion of the bedding plane
* Calculation of sliding of large, potential rock blocks.

Results of Analysis

Calculations have been performed using friction angles of 28°, 30° and 32.5° respectively, and cohesion values 0 and 10 kPa.

Using the above proposed safety factors and different strength parameters, the following anchor capacities are required to stabilise the cut slope.

Strength parameters	ϕ = 28°, c = 0 kPa	ϕ = 28°, c = 10 kPa	ϕ = 30°, c = 0 kPa	ϕ = 30°, c = 10 kPa	ϕ = 32.5°, c = 0 kPa
Anchor load/m cut	885 kN	715 kN	380 kN	210 kN	0

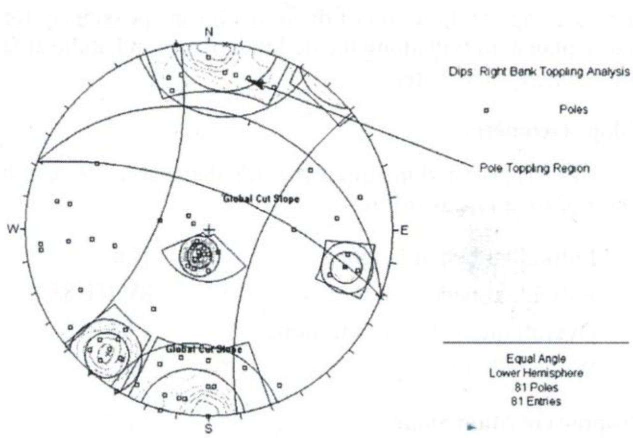

Fig. 4: Toppling failure analysis with DIPS

It seems reasonable to suggest rock anchors at the three lower benches with a working load of 550 kN each, placed at 4 m spacing.

During excavation of the slope, geological parameters such as trace length and friction angle of joints and bedding planes shall be continuously evaluated to verify the chosen design parameters. If the evaluation shows any differences, the design will have to be revised.

The anchors shall have an inclination of 60° from horizontal and length will be 20 to 40 m depending on site conditions.

PSEUDO-STATIC STABILITY OF WEDGES

A pseudo-static stability analysis was also carried out at the permanent slope area for rock wedges using the software SWEDGE. The calculations showed that, for a horizontal earthquake load k_h = 0.21g, the factor of safety was above the required value of 1.05.

SLOPE DESIGN

Design parameters

Direct shear tests performed on bedding planes show low friction angle of around 19-33°, with cohesion value 0 – 10 kPa. Generally, joints have no cohesion. In this case with high effective normal stresses on the bedding planes, the cohesion plays a minor role. For fractured quartzite the basic friction angle is normally around 25-35°. According to the field observations, the joint surfaces are rough to smooth. Therefore, friction angle was presumed to be ~35-40° for an unfilled joint and ~30-35° for bedding planes. As the borehole investigation shows the mica and some loam at some locations on the bedding planes. So, the lower values are considered appropriate for large planar failure analysis.

The information on joint trace length was limited. So, it was presumed to double the available trace length considering the direction of the joints for the possibility for large planar sliding along the bedding plane, and at the area of stepped joint system.

Slope Geometry

The proposed slope geometry for the rock cut slope at the right-bank is as follows:

Individual bench height	12m
Individual bench slope	8V:1H(83^0)
Overall inclination of the slope	~68°
Width of berm	3m

Support of Main Slope

Before excavation of the rock slope starts, two rows of rock bolts of capacity 200 kN shall be provided at a spacing of 2.5 x 2.5 m. The first row of bolts shall be placed 2.5 m above the crest of the cut. The bolts shall 8 m long and inclined at 60^0 from horizontal i.e. into the slope.

To ensure overall stability, pre-tensioned anchors with a capacity of 550 kN shall be placed on the 3 lower benches. The horizontal spacing shall be 4 m. The length of the anchors shall be between 20, 30 and 40 m and the ultimate load is set as 1100 kN (Fig. 5). To transfer the load from the loading plates onto the rock surface continuous reinforced concrete beam shall be constructed.

To take care of possible small sliding and toppling of rock blocks, wiremesh along with 5 m long grouted rock bolts with pattern bolting of 6 m^2 shall be applied. The bolts shall have a capacity of at least 100 kN. The mesh shall be installed as close to the rock surface as possible to collect loose material. After the construction of spillway the remaining gap between the structure and rock cut slope will be backfilled with concrete up to 602.5 msl leaving the upper part exposed, where there is no need of support.

SLOPE DRAINAGE

The slope cut shall be drained by a drainage system. In order to reduce the water pressure, 15 and 25 m deep boreholes shall be drilled in a direction approx. 10^0-15° upwards from the horizontal. The boreholes shall be located at the foot of each bench and at 3 m spacing. At three lower benches, the boreholes shall be 15 m long, whereas at fourth bench (from bottom) the boreholes shall be 25 m long. In every borehole, a slotted PVC pipe wrapped with geotextile shall be inserted. The drainage area for each drainhole shall not exceed 36 m^2.

SLOPE MONITORING

A monitoring system for this type of rock cut should contain devices for the measurements of surface displacements, say, by multiple rod extensometers (MPBX), inclinometers as well as geodetic survey, supplemented by few anchor force measurements by load cells and some piezometers for pore pressure measurements.

Regarding the overall rock mass quality, one extensometer installation per 600 m^2 slope area and one geodetic survey point per 300 m^2 is recommended (Billinger+Berger 2001). For a cut length of 120 m this would require 13 nos. extensometers of 30 m length and 26 geodetic points. Probably a total of 12 extensometers i.e. 4 extensometers placed at each bench number 1, 2 and 4 would suffice. The 26 survey points shall be distributed in a grid system over the entire cut.

Assuming a total number of 120 post-tensioned anchors, 5 to 6 should be equipped with disc load cells for regular inspection of changes in work loads during subsequent excavation, preferably installed at bench 3 (from top).

As the pore pressure in the joints is of great importance to stability, pore pressure measurements shall be performed by piezometers. The length of the piezometers shall be 30 m to 40 m, depending on site condition. The piezometer measurements shall be used to monitor groundwater level and to see whether water table is lowered according to the design or not. If the measurements show that the ground water level is higher than expected, the drainage system has to be improved by means of additional drainage pipes.

CONCLUSIONS AND RECOMMENDATIONS

In overall pit slope, the potential mode of failure is toppling. There is no potential for planar or wedge failure in

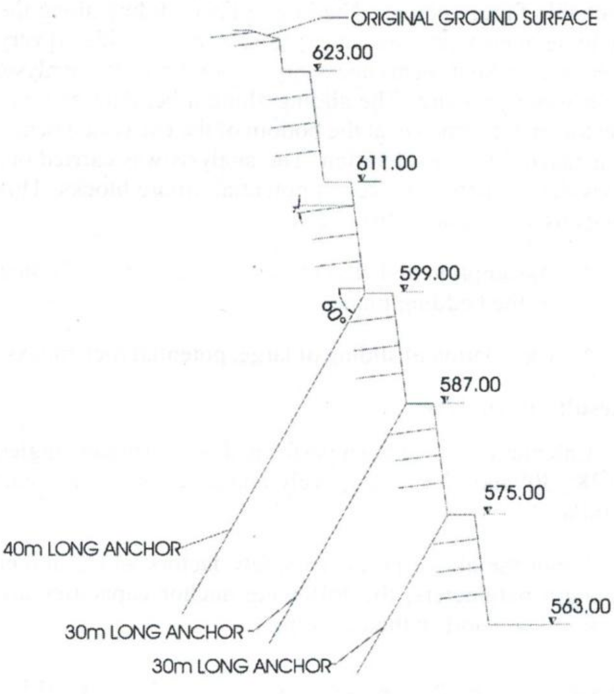

Fig. 5: Typical cross section of cut slope with rock bolts and rock anchors as rock support.

the overall cut slope. By reducing the bench slope angle to approx. 70° (2.7V:1H) the potential of planar and wedge sliding failure is eliminated. The potential of toppling failure remains as for the overall cut slope. A decrease in bench slope angle is not cost beneficial. The increased costs for excavation would be higher than extra costs for supporting the minor wedges.

To take care of both the toppling and the wedge failure, wire mesh together with fully grouted rock bolts shall be used. A bolt shall have 5 m long per 6 m^2. And the proposed slope cut should be reviewed during excavation. For this purpose, a continuous discontinuities mapping of exposed cut surfaces regarding trace lengths, rock bridges and shear strength parameters shall be performed. All the collected data shall be used to review the proposed design. Depending on the results, it may necessary to modify the design of the support or the cut geometry.

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