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Design of Rock Anchoring in a Rock Slope to Achieve the Optimum Design by using Finite Element Method: A Case Study of Slope along Sindhuli-Bardibas Highway

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Abstract

At chainage 17+600 in the section of BP highway, weathered rock slope inclined at 62 degree and 41 meters high, was stabilized with rock anchors. The objective of this study is to redesign the rock anchors to find their optimum orientation by finite element method in order to reinforce unstable weathered rock mass. The anchors are to be extended through the weathered rock mass into stable bed rock. Factor of safety to be achieved against slope failure is taken as 1.5. In this study, an attempt has been made to observe the change in critical SRF with change in inclination of anchors. It is concluded that in this particular slope, maximum FOS is achieved when anchors are placed horizontally.

Rock slope stability analysis with and without anchors based on shear strength reduction method has been performed in Phase2, a finite element software. Traffic loads have also been considered as two point loads. GHB failure criteria is adopted to best define the rock material. Anchor load capacity is based on tensile strength of steel bar and rock-grout bond strength. Results of finite element analysis shows that eight rows of horizontally oriented bar anchors of capacity 340KN with 4m vertical spacing yield a critical SRF of 1.5 and hence sufficient.In this particular slope, horizontal orientation is found to be the optimum orientation of placing anchors.

Keywords

Rock Slope Stability, Finite element method, Critical SRF, Rock Anchor Design

1. Introduction

Rock slopes are encountered in many civil engineering and mining projects. (D. Chamlagain and V. Dangol, 2001) Mai khola rockslide is an example of such failure which involved a combination of plane failure and wedge failure, the major triggering factor being topographic stress, high slope gradient, joint water pressure and river under-cutting at the toe side, causing about 37,500m3 of material to slide. Such slopes can also be invariably found naturally in mountainous areas. These natural slopes, which are stable in their natural state, can fail and become unstable, if disturbed or interfered. Triggering factors for these rock slope failures can be attributed to steep slope gradient, topographic stress, water pressure in joints or change in strength parameters due to weathering. Several countermeasures, broadly classified into four parts as reinforcement, drainage, geometry modification and protection can be introduced. Ground anchoring falls under reinforcement measures, which reinforces the unstable slope by adding stabilizing force by means of bolts or anchors.

[1] Sindhuli-Bardibas highway, constructed with the help of Government of Japan in 1996, encountered similar potential rock slope failure in the highway section at chainage 17+600. in July 2009, a landslide broke through below the road developing cracks on road surface. Owing to the steepness of the rock slope, ground anchor measure was adopted to stabilize the slope and prevent the possibility of rock slope failure in future.

Material model for analysis can be performed by three approaches of modeling rock mass material. One is Continuum interface approach in which the joint sets are implemented explicitly to the intact rock. The type of failure obtained can be planar or wedge type of failure. Another approach is Equivalent-continuum approach in which is modeled as a continuum mass by taking into account the influence of discontinuities on the strength and properties of intact rock. The failure surface obtained by this approach looks more to be circular and global. And the other is Equivalent Continuum-interface approach in which the critical joint set is added explicitly to the above mentioned model by Equivalent-continuum approach.

2. Geological Condition of Study slope

[1]The geological formation in the study location is mostly Quartzite and sandy schist. [1] Talking about discontinuity in schist, the dip angle of schistosity joints is dipping inward the slope. [1] The borehole data of a 30m deep borehole around the study slope shows that there is weathered and fractured quarzitic schist upto a depth of 20m from ground surface and below it there is fresh to slightly weathered and fractured quarzitic schist.The slope accommodates a two lane highway road of width 6.75m.

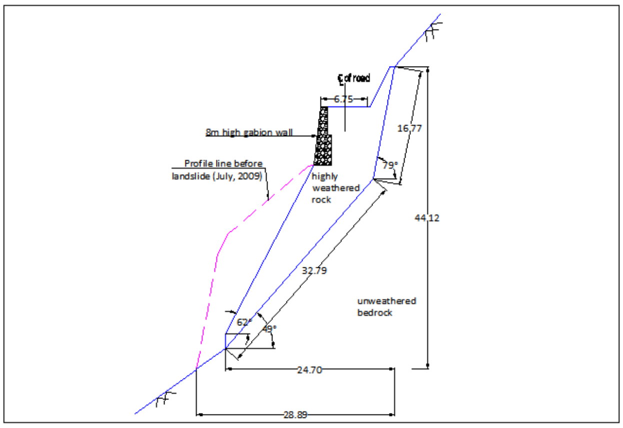


Figure 1: Study Slope

3. Methodology

In this particular study, redesign of rock anchor as reinforcing measure has been done to achieve the optimum orientation and number of anchors at the 41m high, 62 degree steep rock slope at Ch 17+600. The geometry of the slope was according to slope profile produced by JICA. The slope mass has two stratas- weathered rock mass which carries the road and unweathered bed rock as shown in figure. Static loads that are gravity load and surcharge load from vehicles on the road are considered for the analysis.

The anchor arrangement is varied in AutoCAD 2012 and their coordinates are obtained. These coordinates are then input in finite element software to produce required anchor arrangement. The models are run to obtain various critical SRF values.

In the study slope, joints of unweathered quartzitic schist bedrock are dipping inward, and seem to be kinematically stable. The focus is on weathered part of rock slope which is modeled as Equivalent continuum approach, where joints are not separately defined.

[2] According to U.S Department of Transportation, a factor of safety of 1.3 is adequate for low slopes and a factor of safety of 1.5 is required for critical slopes adjacent to major highways.

3.1 Properties of Rock Anchor

A bar anchor may fail in tensile failure of steel, slippage between bar and grout, slippage between grout and rock; or pulling out of the bar with portion of rock mass. The tensile strength of anchor is according to that specified by manufacturing company. For this study, specification of Dywidag threadbar pre-stressing steel as per code ASTM A722 (Grade 150) is used. The specifications of DYWIDAG thread bar Pre-stressing steel (Grade 150) are as mentioned in Table 1. According to the code, maximum allowable temporary test tension shall not exceed 80

Table 1: Properties of Rock Anchor

Bolt Diameter(mm)	26
Bolt Modulus E(KPa)	$2x10^8$
Peak Tensile Capacity (KN)	567
Residual Tensile Capacity (KN)	340
Bond shear stiffness (KN/m/m)	100000
Bond Strength (KN/m)	131.25
Out of plane spacing (m)	3
Pretensioning force (KN)	75% of peak value i.e 425

4. Finite Element Method

Finite element analysis is suitable for correct arrangement of anchors. [3] The limit equilibrium method (LEM) is one of the commonly used methods; however, it only considers the equilibrium of total force, which means that comparison of different anchor positions of the slope is impossible.In the Shear Strength Reduction method, the material strength parameters are progressively reduced by strength reduction factor (SRF) and the finite element stress analysis is performed. This process is repeated for different values of SRF, until collapse occurs and the critical strength reduction factor (critical SRF), or safety factor of the slope is determined. Equivalent Continuum-interface approach in which the critical joint set is added explicitly to the slope model by Equivalent-continuum approach.

4.1 Modeling, Mesh generation and boundary location

Plane strain model has been built with 3-nodded triangular elements. In loading step, each finite element is given a body force (self-weight). Matrix-oriented solution schemes are common for the finite element method. An implicit method is often used for solving step and several iterations are necessary before compatibility and equilibrium are obtained. In this study, Gaussian Elimination is used for solution of the equations. To reach the equilibrium, user defined energy tolerance i.e. 0.001 is used. Analysis of slope model as shown in fig 2 consisted of 4075 no. of nodes and 7426 number of elements.

Boundary condition of model is as shown in fig 2. The location of boundaries in this slope model follows recommendation by [4](Duncan C. Wyllie, 2005) in order to reduce the influence of artificial boundaries on the result. [4](Duncan C. Wyllie, 2005) recommended that the vertical boundary should be at distance greater than width of slope beyond top of the slope, while the horizontal boundary should be at distance greater than half of the height of slope below the toe of the slope. The boundary condition considered is fixed in all directions at the bottom boundary, restrained in horizontal direction at the both sides boundary and slope face is kept free.

The vertices of ends of anchors for various anchor positions across the rock slope are input in Phase2 as per the coordinates acquired from AutoCAD drawing. For instance, table 2 shows vertices for 6 rows of anchors inclined at 15 degree to horizontal such as shown in fig 3.

4.2 GHB Failure Criterion

The strength of rock materials are defined according to Generalized Hoek-Brown Criterion. The GHB criteria is suitable to define strength and failure of rock slope within the rock mass and not along any discontinuity plane. Hence, it is suitable in case of failure within disintegrated and weathered rock mass. While Mohr Coulomb failure criteria is linear, GHB is nonlinear. Hammah et al. (2004) stated that the

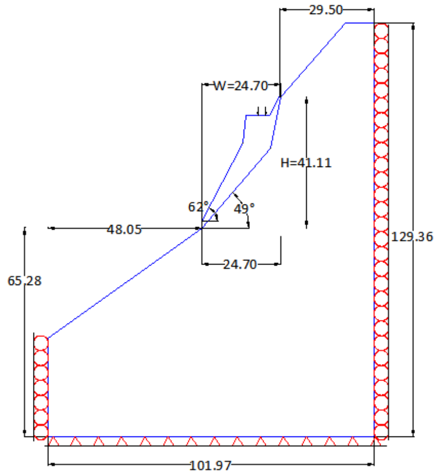


Figure 2: Slope model

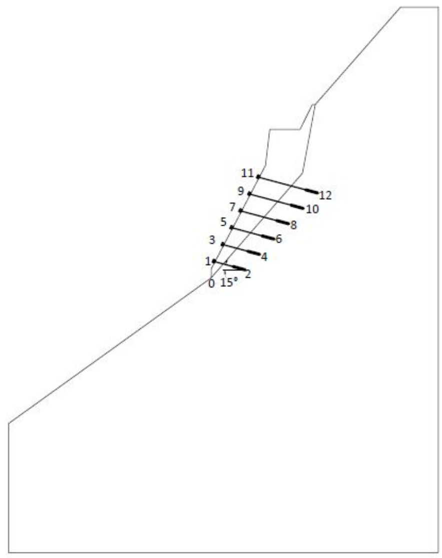


Figure 3: Slope model with anchors

generalized Hoek–Brown criterion is the most suitable strength model for predicting the failure of rock masses, especially in low normal stress ranges. Since, the study slope is not a high slope, the normal stress values must be low.

The Generalized Hoek-Brown criterion is an empirical criterion which establishes the strength of rock in terms of major and minor principal stresses by equation 1.

$$\sigma_1 = \sigma_3 + \sigma_{ci}(m_b \frac{\sigma_3}{\sigma_{ci}} + s)^a \quad (1)$$

where,

$$m_b = m_i \exp(\frac{GSI - 100}{28 - 14D}) \quad (2)$$

Table 2: Coordinates of anchor ends corresponding to figure 3 to be input in Phase2

Points	Vertices	Points	Vertices
0	(0,0)		
1	(0.91,3.84)	7	(7.21, 15.84)
2	(8.09,1.92)	8	(18.4, 12.84)
3	(3.01,7.84)	9	(9.31, 19.84)
4	(11.53,5.56)	10	(21.84, 16.48)
5	(5.11, 11.84)	11	(11.42, 23.84)
6	(14.97, 9.2)	12	(25.28, 20.13)

Table 3: Input parameters for rock mass

$$s = \exp(\frac{GSI - 100}{9 - 3D}) \tag{3}$$

$$a = \frac{1}{2} + \frac{1}{6}[\exp(\frac{-GSI}{15}) - \exp(\frac{-20}{3})] \tag{4}$$

5. Properties of rock mass

The properties of weathered rock mass and unweathered bedrock adopted in analysis are as given in table 4 and 5 respectively.

Table 4: Properties of weathered rock mass

Parameters	Value
Unit weight	23 KN/m^3 [1]
Intact rock strength	25 MPa
Geological Strength Index GSI	20

6. Design anchor load

The capacity of anchor is the minimum of tensile strength of anchor steel bar and pull-out capacity.

For 1” dia Dywidag pre-stressed threadbar anchor as per ASTM A722 (Grade 150), ultimate tensile capacity of anchor = 567 KN

Design load of anchor= 60% of ultimate tensile capacity of anchor

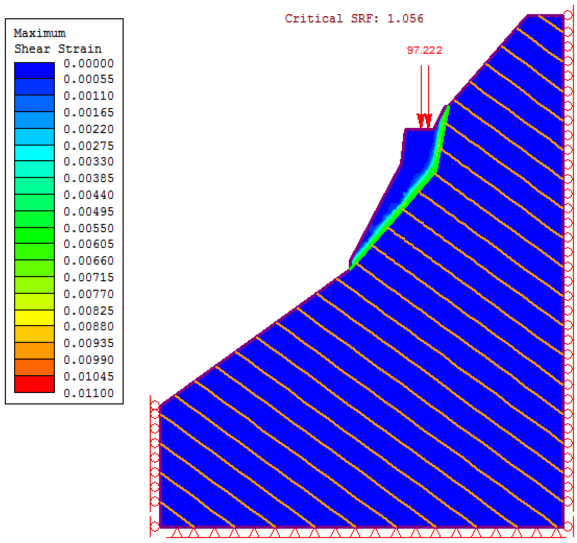
Therefore, design load of each anchor (P) = 60% of 567 KN= 340 KN

Pull-out capacity of anchor is the minimum of grout-rock bond strength and force required to pullout rock by shear. However due to lack of pullout test, it is

Table 5: Properties of unweathered bedrock

Parameters	Value
Unit weight	23 KN/m^3 [1]
Intact rock strength	60 MPa
Geological Strength Index GSI	70

Figure 4: FEM SSR result



taken to be equal to grout-rock bond strength. The rock-grout bond strength depends on type of rock and grout material. [5](Berardi, 1972) For cement-grout and quartzite/schist rock, the recommended value of working bond stress is 1.1 MPa.

Taking grout diameter=38mm,
Working Grout-rock bond capacity per m grout length
= working bond stress x perimeter of grouted hole
= 131.25 KN/m

Since design anchor load = 340 KN,
Required grout length L = 340/131.25= 2.59m smaller than 3m
Therefore, design grout length L = 3m

7. Results and Discussion

Initially, rock slope stability was analysed without anchoring. It resulted critical SRF value i.e FOS to be 1.056 which is not sufficient to ensure stability of the slope and requires stabilizing measure. The result is as expected. The probable failure surface is represented by green zone in fig 4. It lies within the weathered rock mass near the interface between the weathered rock mass and bedrock.

Now, when adding tieback anchors, length of which are given in table 6, oriented horizontally in 8 rows through the unstable weathered rock mass into the bedrock, results in critical SRF of 1.5 as shown in fig 5

Figure 5: FEM SSR result: Rock slope with horizontal anchors

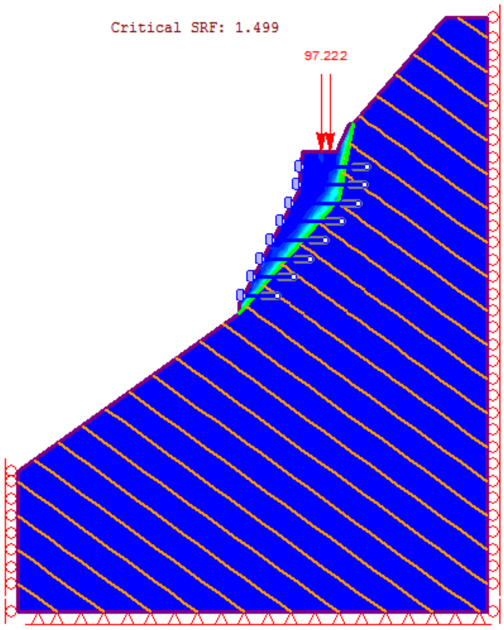


Table 6: Final design length of anchors

Sequence of anchor	Total Length (m)	Fixed Length(m)
1st row from bottom	7.44	3
2nd row from bottom	8.82	3
3rd row from bottom	10.2	3
4th row from bottom	11.61	3
5th row from bottom	12.97	3
6th row from bottom	14.35	3
7th row from bottom	14.17	3
8th row from bottom	14.44	3

7.1 Effect of orientation of anchors on FOS

The effect of change in orientation of anchors in the study slope was studied in finite element software. For this purpose, 6 rows of anchors on lower part of the slope have been used at 0, 5, 10, 15, 20 and 25 degree. Some of them are shown in fig 6 and 7.

It is observed that the critical SRF value is gradually decreasing with increase in inclination of anchors with horizontal as shown in fig 8.

Maximum SRF is achieved when anchors are placed horizontally. This is probably because maximum

Figure 6: Result: Anchors oriented at 20 degree

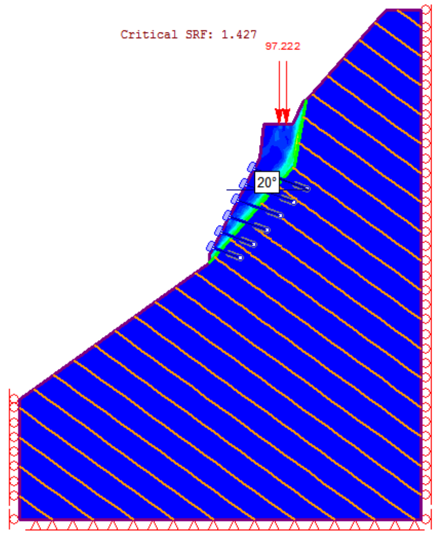
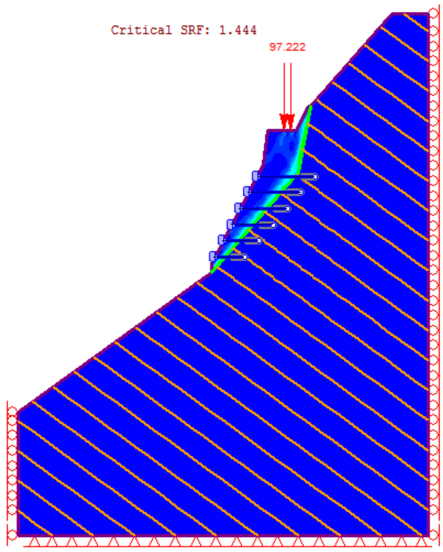
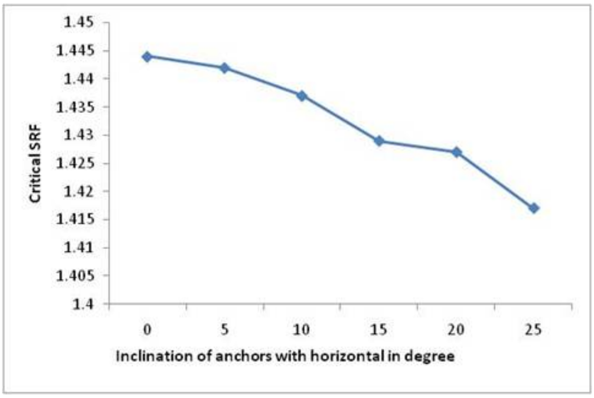


Figure 7: Result: Anchors oriented horizontally



shear resistance along failure plane is developed when the force acts horizontally. This result can be verified analytically by considering the effect of anchor force which is 340KN max in the form of a force of 340KN magnitude acting on a failure plane. As explained above, the failure is likely to take place along the interface between weathered rock and bedrock. As shown in fig 1, there exist two planes of interface. The lower plane is inclined at 49 degree. Assuming planar failure along lower plane, for an angle of internal friction ¡ 54 degree and failure plane inclined at 49 degree, plane failure analysis considering an anchor force of magnitude P determines that maximum resisting force along the failure plane is achieved when the force acts horizontally. When the force acts at any angle steeper than horizontal and flatter than

Figure 8: Critical SRF with Vs inclination of anchors



normal to the plane, the resisting force is less.

8. Conclusion

The study conducted stability analysis of a 41m high rock slope with slope inclination of 62 degrees consisting of highly weathered schist rock having unit weight 23 KN/m^3 with intact rock strength 25MPa, GSI value 20. The probable failure plane is obtained to be along the interface of the weathered rock mass and stable bedrock; it is inclined at 49 degree. 8 rows of horizontally oriented bar anchors each with a capacity of 340KN is required to stabilize the slope with a factor of safety of 1.5. For this particular slope, the optimum orientation of anchor is found to be horizontal; steeper the anchors, less the factor of safety.

Future Enhancements

It is suggested that in designing of rock anchors, the failure of rock anchors due to pulling out of rock mass shall be considered which could be studied from pull out test results.

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